UKOOA DECOMMISSIONING COMMITTEE

DRILL CUTTINGS INITIATIVE - RESEARCH & DEVELOPMENT PROGRAMME - PHASE II

REPORT ON TASK 5B

IN SITU SOLUTIONS: COVERING

Report No. 238.UK.0700 - Rev2A
January 2002

Dredging Research Ltd

Bargate House, Catteshall Lane,
Godalming, Surrey GU7 1LG, UK
Tel: +44-1483-860731

In association with:

Centre for Environment, Fisheries & Aquaculture Science (CEFAS)
ERM (Scotland) Ltd
Galbraith Consulting Ltd

Supported by:

Tideway bv **Boskalis Offshore bv** **Van Oord ACZ bv**

UKOOA DECOMMISSIONING COMMITTEE

DRILL CUTTINGS INITIATIVE - RESEARCH & DEVELOPMENT PROGRAMME - PHASE II
TASK 5B - IN SITU SOLUTIONS: COVERING

EXECUTIVE SUMMARY

The Phase I preliminary review of *in situ* covering of drill cuttings concluded that the option was practical and that covers could be constructed using natural granular materials such as sand, gravel and armour stone. This Phase II study has been undertaken in order to identify the limitations of the technique.

This Phase II assessment has confirmed that covers can be placed using proven construction methods. They should comprise an initial layer of sand followed by a gravel filter layer and an outer protective layer of armour stone. Alternative materials, such as membranes, mattresses, concrete and various resinous and bituminous formulations would be costly and very difficult to place. They have few, if any, advantages over natural materials. However, at present, there is no proven method of construction underneath existing installations although it is likely that methods could be developed. Covering may not be appropriate for concrete installations that are fully removed but is a practical option for fully and partly removed steel structures and, possibly, partly removed concrete structures.

Uncertainty about the *in situ* properties of drill cuttings prevents confident definition of the geotechnical constraints to covering. More work is required in this area but indicative designs have been developed on the basis that some piles may have a low factor of safety against instability. These envisage the need to ‘build-out’ steep concave pile slopes and to reduce the slopes of steep conical piles by varying the thickness of the initial sand layer to ensure pile stability and to provide a suitable slope for armour stone of a size which can be placed using existing plant. Alternatively, the tops of some piles might be removed prior to covering.

The armour layer will provide adequate short term protection against the impacts of severe storms, trawling and collapse of parts of partially-removed structures but it is not practical to provide guaranteed long-term protection against the cumulative effects of trawling, emergency anchoring by large vessels and repeated structure collapse events, particularly in the case of part-removed concrete structures. Monitoring and maintenance will therefore be required, the details of which, particularly the required duration, can only be defined when other studies have determined the extent to which the cuttings must be isolated, taking into consideration possible time-dependent changes of the nature and degree of contamination.

The sand layer can be designed to accommodate all the pore water that will be expelled from the cuttings due to consolidation. In the long term, contaminants will be released due to chemical migration and by pore water exchange caused by a pumping action induced by waves but these losses are predicted to be negligible. Potentially more significant, short-term releases may occur during construction due to cuttings disturbance when placing the initial sand layer. Leachate analyses and ecotoxicology tests using samples recovered from the Ekofisk and Beryl A piles, in combination with estimates of potential rates of release during construction, indicate that dilution of the released leachate will be rapid and that the risk of acute or chronic toxic effects will be very low. Limited data from other cuttings piles suggests that this conclusion may apply to a majority of piles but additional analyses and diffusion modelling are required to increase the knowledge database before firm conclusions can be drawn concerning the number of piles for which this finding is valid.

A risk review has concluded that, as long as the cuttings are physically isolated for the period during which they pose a threat to the environment, covering is a generally low-risk option which can be achieved using proven methods of construction and with little or no adverse impact on the marine environment. However, adoption of this option represents a potential liability due to the possibility of damage of the cover resulting in release of contamination and a risk to fishing activities. Covering may be viewed adversely by the general public and stakeholder groups such as fishermen’s associations.

UKOOA DECOMMISSIONING COMMITTEE

DRILL CUTTINGS INITIATIVE - RESEARCH & DEVELOPMENT PROGRAMME - PHASE II TASK 5B - IN SITU SOLUTIONS: COVERING

CONTENTS

7 GEOTECHNICAL ASPECTS OF COVER DESIGN

8 CONTAMINANT LOSS THROUGH PILE COVERS

9 ECOLOGICAL IMPLICATIONS OF COVERING

13 SUMMARY OF CONCLUSIONS - CONTINUED

UKOOA DECOMMISSIONING COMMITTEE

DRILL CUTTINGS INITIATIVE - RESEARCH & DEVELOPMENT PROGRAMME - PHASE II

TASK 5B - IN SITU SOLUTIONS: COVERING

1 INTRODUCTION

1.1 BACKGROUND

The United Kingdom Offshore Operators Association (UKOOA) has commissioned an R & D programme to investigate a range of issues concerning the drill cuttings which have accumulated under and around fixed installations in the Central and Northern Sectors of the North Sea. Phase I of the programme was completed in early 2000 and included a review of the practicality of covering drill cuttings *in situ* (ERM, 1999[1]). Covering of contaminated sediments, either *in situ* or after dredging and placement in specially-constructed pits, has been adopted in several countries, notably the USA and Hong Kong, but has yet to be applied to drill cuttings. The main conclusion of the Phase I review was that *in situ* covering is viable in both engineering and environmental terms. A conceptual cover design was proposed which comprised a layer of sand intended to accommodate contaminated pore water expelled from the cuttings as they consolidate. The sand is protected by layered armour comprising gravel overlain by armour stone.

Phase II of the Drill Cuttings Initiative commenced in August 2000. Task 5b comprises a more detailed evaluation of the *in situ* covering option. This report has been prepared by Dredging Research Ltd (DRL) in association with the Centre for Environment, Fisheries and Aquaculture Science (CEFAS) and Environmental Resources Management (ERM). The study team includes Tideway BV, Boskalis Offshore BV and Van Oord ACZ BV who are leading dredging and offshore civil engineering contractors with expertise in underwater placement of natural and artificial materials over and around seabed installations.

The overall objective of Task 5b, as defined in the Contract Terms of Reference (CTR), is to determine the limitations of relevant covering options on a range of pile types for effectiveness against disturbance. The CTR specifically requires the evaluation of the following topics:

Cover design, including:
- loss of contaminated leachate;
- impacts of trawling, resistance to trawling;
- impact of storms;
- minimum thickness;
- construction materials;
- maximum gradient over which a cover would be stable.

Design Implementation
- Very long term stability – integrity warranty for suggested designs

Cover construction:
- What would be the impact of jacket removal prior to covering?
- How would the cover be constructed?
- Time, costs and resource consumption for implementation.

[1] *Environmental Resources Management (1999).* The Practicality of Covering Drill Cuttings In Situ. *Report to UKOOA under Task 5.1 of Phase 1 of the UKOOA Drill Cuttings JIP.*

Integrity and Monitoring:
- How would you know it is working over the long term?
- What kind of monitoring would be required?

1.2 STRUCTURE OF THIS REPORT

The general structure of this report is summarised below:

Section 2 comprises a review of the nature and occurrence of cuttings piles and summarises their geotechnical properties using data obtained during previous field and laboratory investigations. The results of geotechnical tests undertaken during this study are presented.

Section 3 discusses the factors which must be considered when designing pile covers.

Section 4 reviews potential materials and methods of construction and identifies viable options.

Section 5 presents data on waves and currents in the Northern and Central sectors. These are analysed to provide input data for the design of cover protection and for the assessment of delays to cover construction.

Cover protection requirements are examined in *Section 6*. Protection against extreme environmental conditions, anchoring, trawling and the collapse of any remaining parts of decommissioned installations are considered in turn. Potential impacts on the cuttings pile during structure decommissioning are also reviewed.

Section 7 deals with the geotechnical aspects of cover design, focusing mainly on pile stability during and following covering. Estimates of the rates and magnitudes of consolidation are derived leading to estimates of potential rates of contaminated leachate release.

The potential impacts of contaminant loss through permeable pile covers are reviewed in *Section 8*. The review is based on leachate analyses and toxicology tests undertaken during this study on three samples recovered from the Ekofisk and Beryl A cuttings piles.

Section 9 discusses the ecological implications arising from the presence of the cover pile and assesses options for habitat enhancement.

Future monitoring and maintenance requirements are discussed in *Section 10*.

Section 11 deals with the execution of example covering operations, presenting cost and energy budget estimates for pile covering based on four base case cuttings piles.

An overall assessment of the potential risks arising from in situ covering is presented in *Section 12*, dealing in turn with strategic and operational issues, environmental matters and health and safety.

The conclusions of this study are collated and summarised in *Section 13*.

1.3 AUTHOR'S NOTE

The authors of this report are aware that it will be made available to a wide readership with, inevitably, varying degrees of familiarity with some of the technical subjects that are reviewed. In some sections, particularly *Section 7* (geotechnical constraints), the authors have therefore discussed some topics at a level of detail that is greater than might normally be presented in a technical report addressed to a more restricted readership. It is hoped that those who already possess detailed knowledge of these topics will understand the need for this approach.

2 CHARACTERISTICS OF CUTTINGS PILES

2.1 INTRODUCTION

In order to develop designs for pile covers, it is necessary first to define the scope of the problem. This section provides an overview of the types of structure under and around which cuttings piles occur, the dimensions and shape of the piles, and their constituents and geotechnical properties.

2.2 OFFSHORE INSTALLATIONS - DEFINITIONS AND STRUCTURE TYPES

An installation refers to any facility or group of adjacent facilities, with the exception of pipelines, that generally stays in one place during production or exploration operations. Hence an installation may be fixed to the seabed, mobile (floating or standing on the seabed) or subsea. This study is concerned with the cuttings piles that have accumulated under and around fixed steel and concrete platforms. A platform comprises the whole of a fixed entity which projects above sea level, including the substructure (between the seabed and the topsides), the topsides structure and the equipment within and upon the topsides. In this context ‘fixed’ means that the main substructure is fastened to the seabed by piling or by gravity foundations.

2.2.1 Fixed steel structure (or Jacket)

A conventional fixed steel structure comprises a framework of tubular steel which, in the North Sea, normally has between 4 and 8 legs and is piled to the sea bed through the legs and/or through pile sleeves adjacent to the legs. *Figure 2.1* shows a typical fixed steel structure. There is a single example (Maureen) of a fixed steel structure that is held to the seabed by gravity rather than by foundation piles. The weight of such structures in the North Sea (excluding the foundations and topsides) ranges from several hundred tonnes to about 40,000 Tonnes.

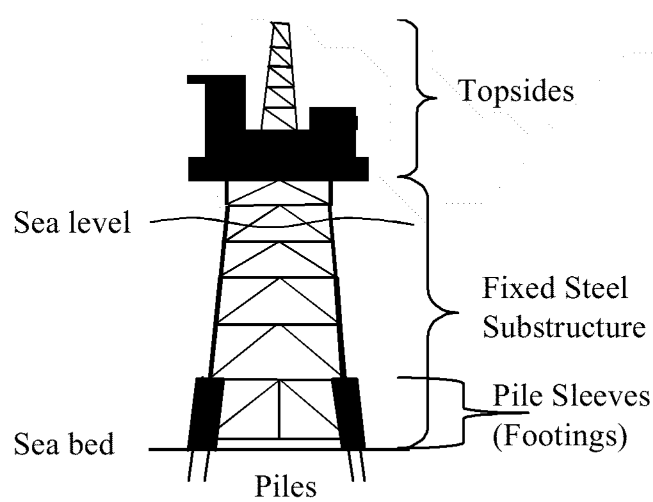


Figure 2.1. Simplified elevation of a fixed steel structure

The drill cuttings associated with fixed steel structures have accumulated under and around the installation. In the majority of cases where data are available, the highest part of the cuttings piles are located within or very close to the substructure. The piles may cover part or all of the drilling template and some of the lower horizontal structural members and may envelope the relatively complex footings structures.

Figure 2.2 is the plan of a typical fixed steel structure at the level of the lowest horizontal frame showing the complexity of the structure which may be part-buried by the cuttings. In addition, there may be pipelines and service cables leading to the structure that are covered by cuttings.

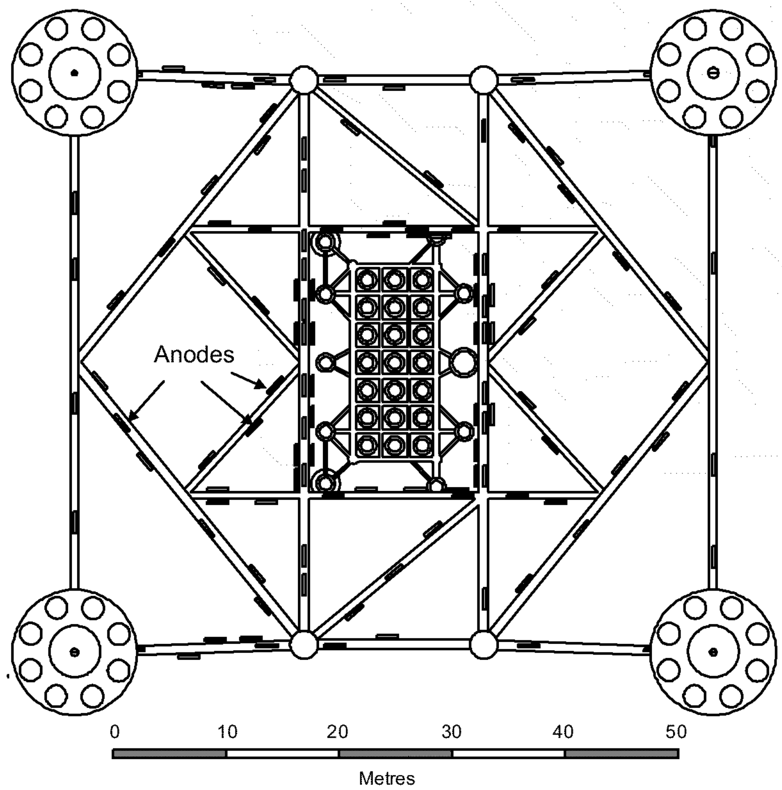


Figure 2.2 Plan view of a typical fixed steel structure at level of lowest framework

2.2.2 Fixed concrete platform

There are 24 concrete platforms in the North Sea. They vary in arrangement but are characterised by their considerable weight (150,000 to 1,500,000 Tonnes in the Northern North Sea), by having gravity foundations (ie. they rest on the seabed without being piled), and either one central core rising to support the topsides or between 2 and 4 legs rising from a concrete cell structure at the base to support the topsides. Many of the multi-leg concrete structures have used the cells at the base for storage, and even for separation of hydrocarbons. Typical concrete platform arrangements are shown in *Figure 2.3*.

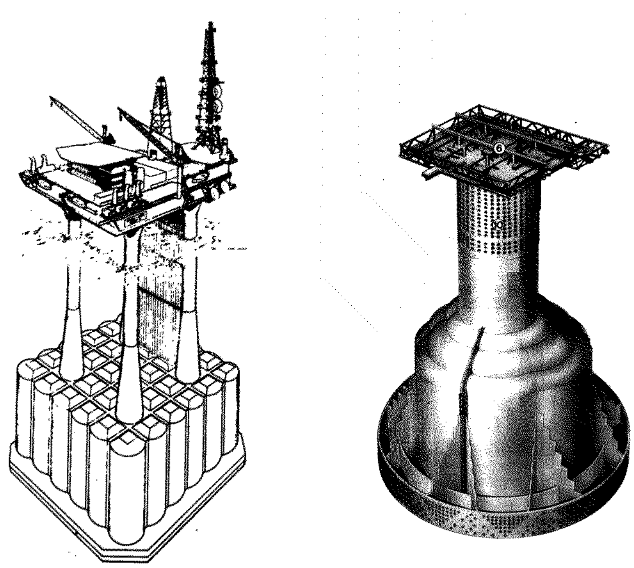


Figure 2.3 – Typical Fixed concrete platforms

Drill cuttings may lie on top of the gravity base and on the adjacent seabed. The cuttings on the seabed tend to be banked up against one or two sides of the base depending of the location of the cuttings discharge caisson and the current regime. Those on top of the base are generally offset from the centre and may comprise a significant proportion of the total volume of cuttings. Where the base is formed from contiguous cells, cuttings may infill the voids between cells (*tri-cells* or *star cells*).

2.3 OSPAR '98 DECOMMISSIONING REQUIREMENTS AND OPTIONS

The OsPar 98/3 decision on the requirements for removal and disposal of offshore oil and gas installations in their waters is summarised in *Figure 2.4*. The intent is full removal and reuse or onshore disposal of all installations, however derogations are possible for parts of certain fixed steel structures, for concrete structures and for some foundation elements of floating structures. The OsPar '98 deliberations did not address drill cuttings.

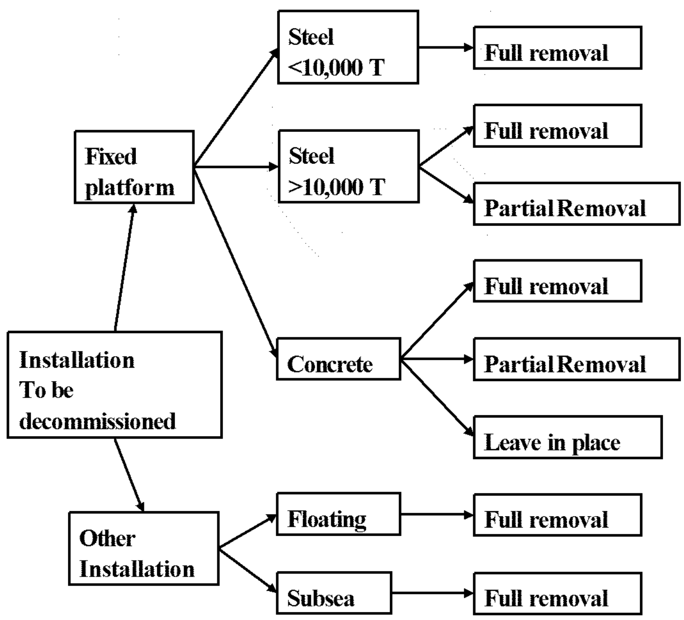


Figure 2.4 – Decommissioning options for installations with drill cuttings piles

The type of structure and the timing of its removal relative to the covering of the cuttings pile are important considerations with respect to methods of covering and the requirement to protect cuttings piles. These aspects are reviewed later in this report. At this stage, it need only be noted that:

- pile covers may need to be constructed in situations where a substantial part of the structure remains in place;
- in some cases, structure removal will require displacement or removal of parts of the cuttings piles in order to gain access to the structure and to connected pipelines and other services.

2.4 CUTTINGS PILES

2.4.1 Water depths

Water depths at cuttings pile locations (*Figure 2.5*) in the UK Northern Sector range between 100 metres and 186 metres (Magnus). In the UK Central Sector, water depths range between 46 and 150 metres although only three sites (Beatrice AD, B and C in the Moray Firth) lie in less than 70 metres of water.

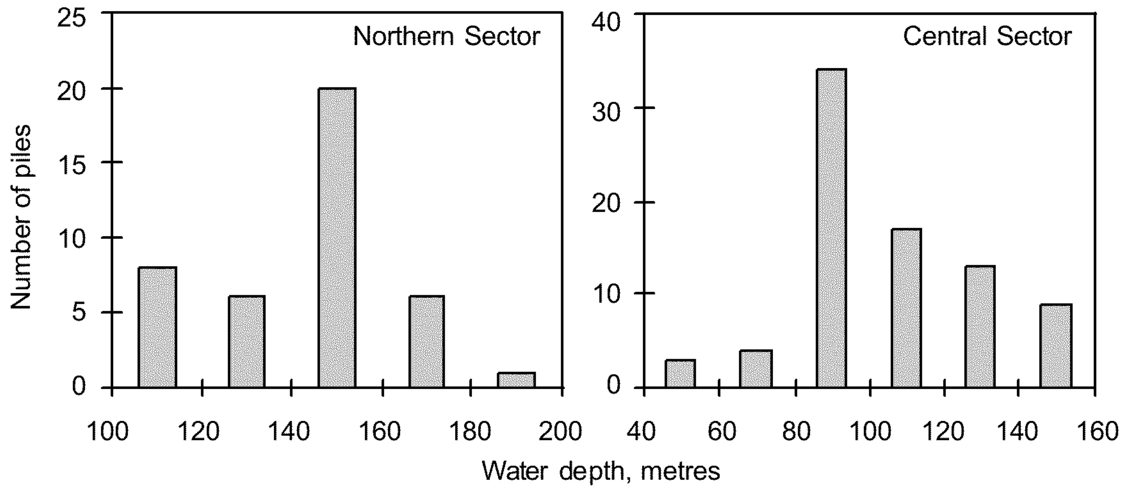


Figure 2.5. Water depths at cuttings pile locations in the Northern and Central Sectors

2.4.2 Pile volumes

Cordah (1998[1]) have collated data on measured pile volumes and have estimated volumes for some piles where measured volumes are not available. These are summarised in Figure 2.6.

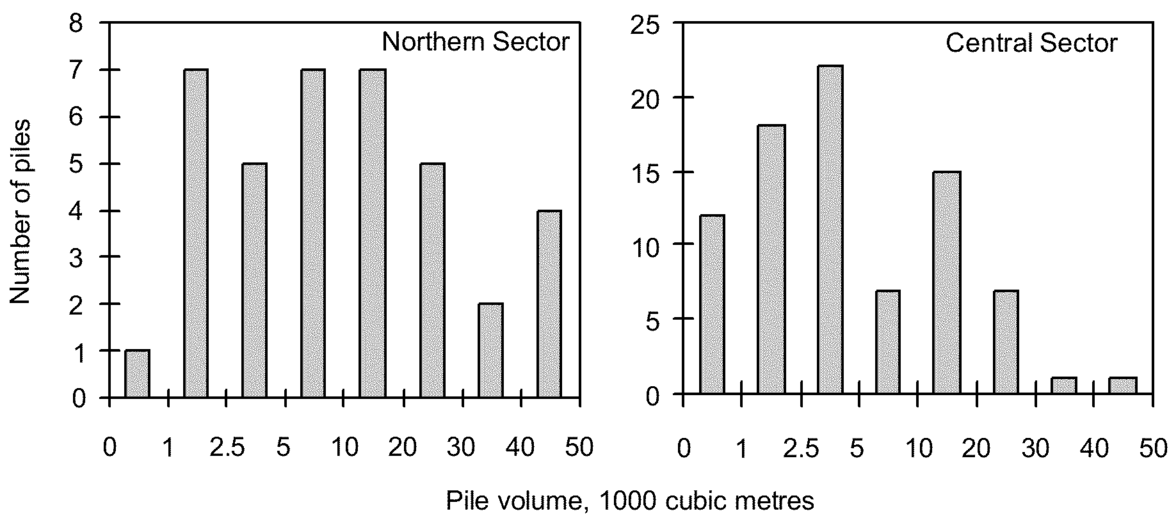


Figure 2.6. Pile volumes in the Northern and Central Sectors

2.4.3 Pile dimensions

Summary data on the areas and heights of cuttings piles are provided by Cordah (*Op cit.*) and Britsurvey (1998[2]). The areas of the cuttings piles (excluding concrete installations) are compared with volumes in *Figure 2.7* from which it is evident that there is considerable scatter in the expected trend of area increasing as volume increases.

Although limited, the Britsurvey data suggest that most cuttings piles under steel structures are very approximately sub-circular in plan. The centre of the pile may be offset to one side or another of the main structure, depending mainly of the location on the cuttings discharge caisson and the tidal current regime but usually lies within, or very close to, the structure footprint.

[1] Cordah, 1999. *Determination of the Physical Characteristics of Cuttings Piles, using Existing Survey Data and Drilling Information*. Report for the UKOOA Drill Cuttings Joint Industry Project, Cordah Limited, Aberdeen, November 1999.
[2] Britsurvey, (1998). *Drill Cuttings Survey 1997*. Report for Shell U.K. Exploration and Production. Britsurvey, Great Yarmouth, March 1998.

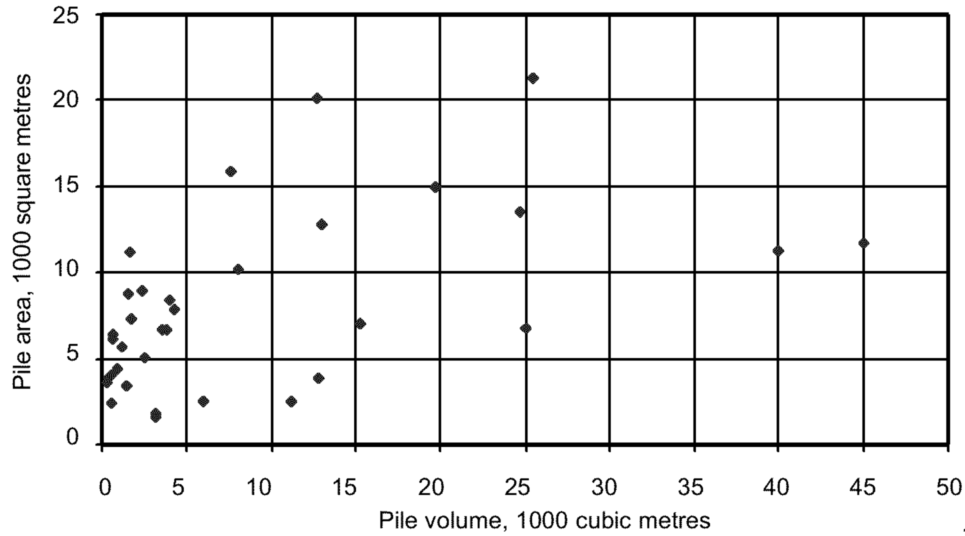


Figure 2.7. Pile areas plotted against pile volumes in the Northern and Central Sectors

The Britsurvey report includes only one pile (Auk A), out of a total of 12, which is markedly elongate. This pile is located in the southern part of the Central Sector. It has a long axis, aligned with a strongly bi-directional tidal current, of about 200 metres and a short axis of about 50 metres. Pile height data are plotted against volume in *Figure 2.8*. As with area, there is a trend of height increasing with volume but with a high degree of scatter.

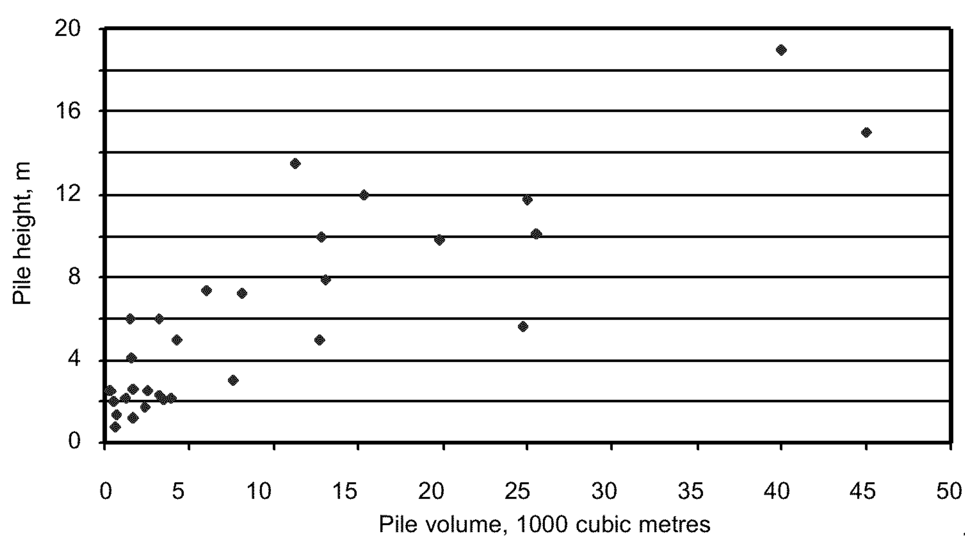


Figure 2.8. Pile heights plotted against pile volumes in the Northern and Central Sectors

There are limited data on the slopes of the piles. Britsurvey (1998, *Op cit.*) surveyed 12 piles using a swathe system and found maximum pile slopes of between 6^{o} and 26^{o} (*Figure 2.9*, left). Four piles were located under concrete structures and, in two of these cases, heights and angles were reported separately for the piles lying on the seabed and those lying on top of the gravity base. A comparison of maximum slope angles and heights (*Figure 2.9*, right) reveals a very weak trend in which the steepest slopes tend to be associated with the highest piles.

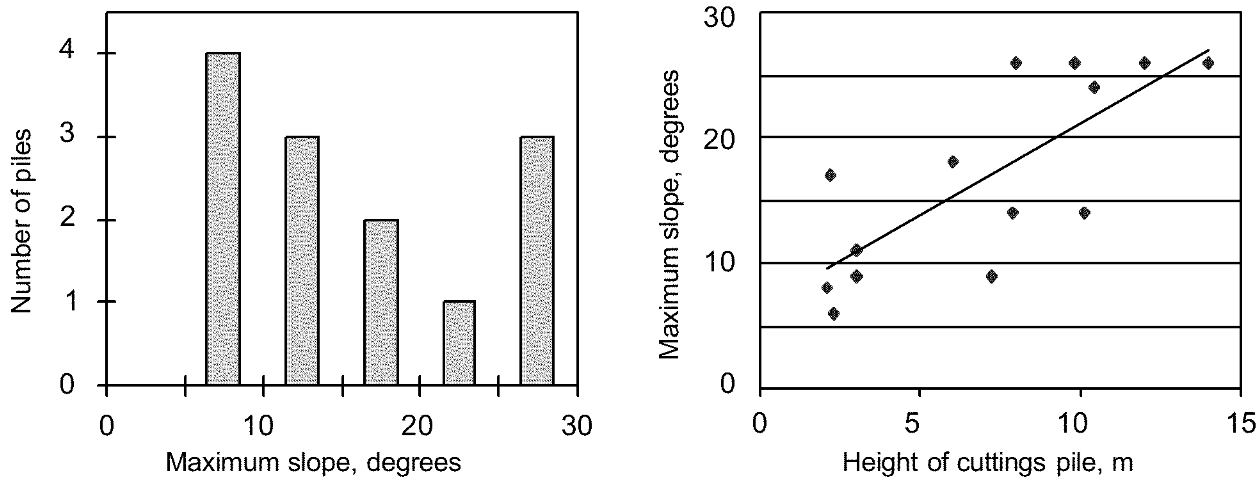


Figure 2.9. Left: distribution of observed maximum slope angles, Right: slope angle plotted against pile height.

This trend may be explained by the fact that site conditions which favour the formation of high cuttings piles (*ie.* relatively weak currents and cuttings discharge from a low altitude above seabed) are likely also to permit cuttings accumulations with relatively steep slopes. On this basis, and noting that the highest pile (Heather) is about 19 metres, it may be surmised that maximum slopes are likely to be of the order of 35°. Such an angle agrees with expectations based on geotechnical knowledge of slopes formed in loose granular deposits. However, in a general discussion of visual and acoustic surveys at 21 piles Altra (1996[1]) noted that the angle of repose varied but was generally less than 40°.

A detailed survey at Beryl Alpha (Subsea Offshore, 1996[2]) showed the pile to comprise a semi-circular mound with a radius of about 65 metres banked against the concrete base of the structure. The upper part of the concave slope is typically at about 40° with maximum slopes (over distances of one or two metres) as high as 60°. The angle of the lower, peripheral slope varied between 5° and 15°.

In an attempt to gain further insight into pile morphology, the more numerous estimates of pile volume, area and height provided by Cordah have been used to derive the following ratio for piles under steel structures:

$$\frac{\text{area x height}}{\text{volume}}$$

In the case of a perfect conical pile, this ratio (or *shape factor*) should have a value of 3. *Figure 2.10* shows the shape factors of 31 piles plotted against pile volume. Only six piles have a ratio of about 3. The remainder have greater ratios suggesting that most piles have markedly concave slopes near their centres with wide fringing areas where the slopes are relatively shallow. It is very noticeable that piles with volumes of less than 2,500 m^3 have the highest ratios. For example, the Gannet C2 pile has a reported volume of 252 m^3 and an area of 3,592m^2. If it were a regular cone, the height would be 0.2 m but the reported height is 2.5m. An explanation for this may be that cuttings will tend to be supported or retained by parts of the structure, particularly the template and legs. This may give rise to distinct, but relatively small pinnacles on an otherwise very low cuttings pile with gentle slopes. This effect will be less pronounced in larger cuttings piles which will tend to overwhelm structures such as the template.

[1] ALTRA, 1996. *Review of Drill Cuttings Piles*. Report UKO.872. Rev – 1 to UKOOA/DTI, ALTRA Safety and Environmental Ltd
[2] Subsea Offshore Ltd., 1996. *Mobil Beryl Field, 1996 Sonar Graphics Survey Results*. Report No AB-R-RP-00132 to Mobil North Sea, Subsea Offshore Ltd., Aberdeen, November 1996.

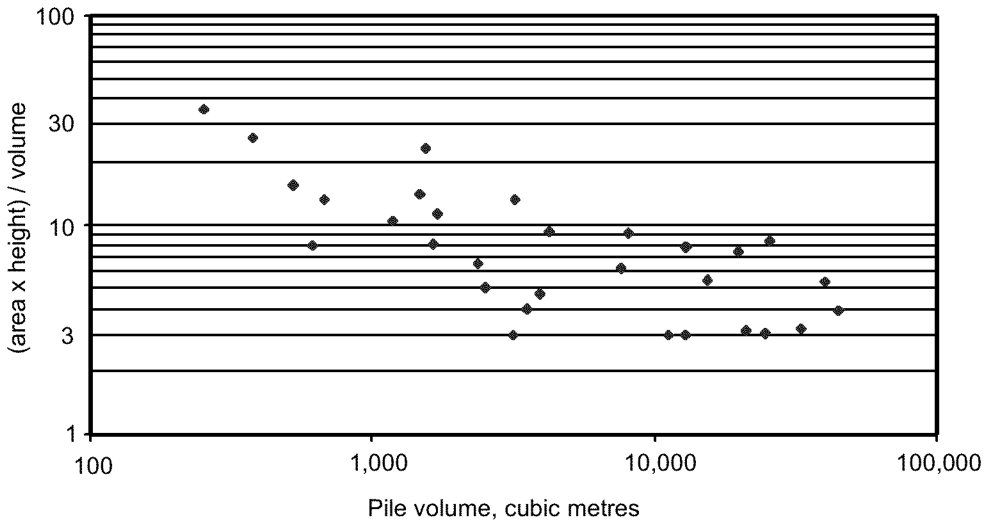


Figure 2.10. Pile shape factors plotted against pile volumes in the Northern and Central Sectors

Taken together, all of these data suggest that the morphology of cuttings piles can be summarised as follows:

1) in very broad terms, the larger cuttings piles tend to exhibit the steepest overall slopes;
2) pile slopes are typically concave with maximum angles occurring in the upper part of the slope;
3) for larger piles, the angle of the upper slope may be up to 35-40^{o} but steeper slopes (up to 60^{o}?) may occur locally over short distances; the lower slopes lie at angles of between 1^{o} and 10^{o} except at the outermost fringe where slopes will be imperceptible as they merge with the surrounding seabed;
4) for piles of less than about 2,500 m^3, the reported heights appear likely to represent relatively minor features where cuttings are supported or restrained by parts of the structure and probably exhibit steep slopes; overall, these piles are expected to comprise low mounds with shallow slopes.

2.4.4 Example pile morphologies

The review of pile dimensions can be used to derive 'normalised' pile morphologies to investigate geotechnical design constraints and for the assessment of cover designs and construction costs. The available data indicate that pile shapes can vary between almost perfect cones and markedly concave forms in which there is a distinct central pinnacle surrounded by a shallow slope. All of the forms between these extremes can be approximated by two superimposed cones (refer to *Figure 2.11*).

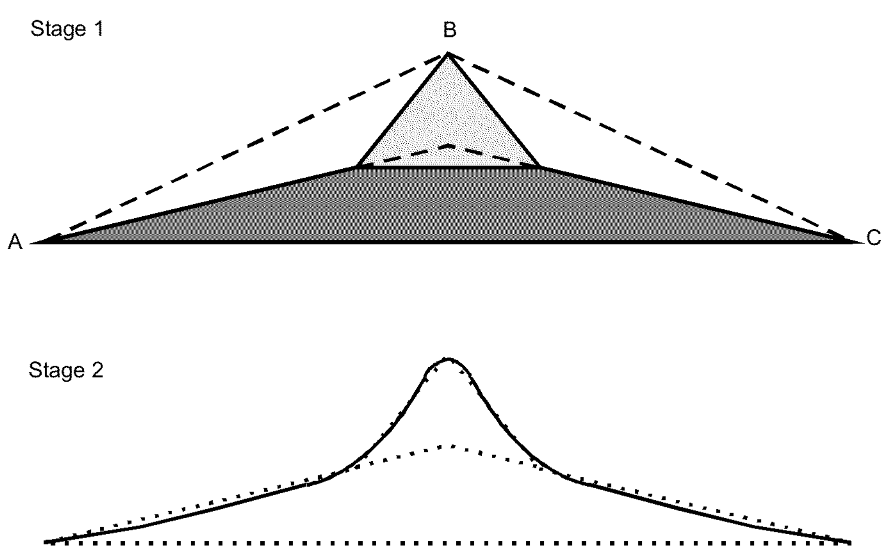


Figure 2.11 Derivation of normalised pile morphologies.

Stage I of the normalisation process involved determining the dimensions of the two cones for each of the 31 piles for which volume, area and height data are available. In doing this, two simple rules were applied:

1) the slope of the lower cone had to be as steep as possible but no steeper than the theoretical slope between the outer edge of the pile to the peak (line A-B in *Figure 2.11*);
2) the slope of the upper cone had to be as steep as possible but no more than 37.5^o (an assumed 'steep slope').

Stage II of the normalisation simply involved drawing a smooth curve through the two cones to represent a cross section through the normalised pile.

Steel Structures – Large Piles

The data in *Figure 2.10* suggest that large cuttings piles (here defined as being >2,500 m^3) under steel structures may vary between regular cones (shape factor = 3) with relatively uniform slopes and markedly concave piles (shape factor = 10). This leads to three example piles that are illustrated in *Figure 2.12*.

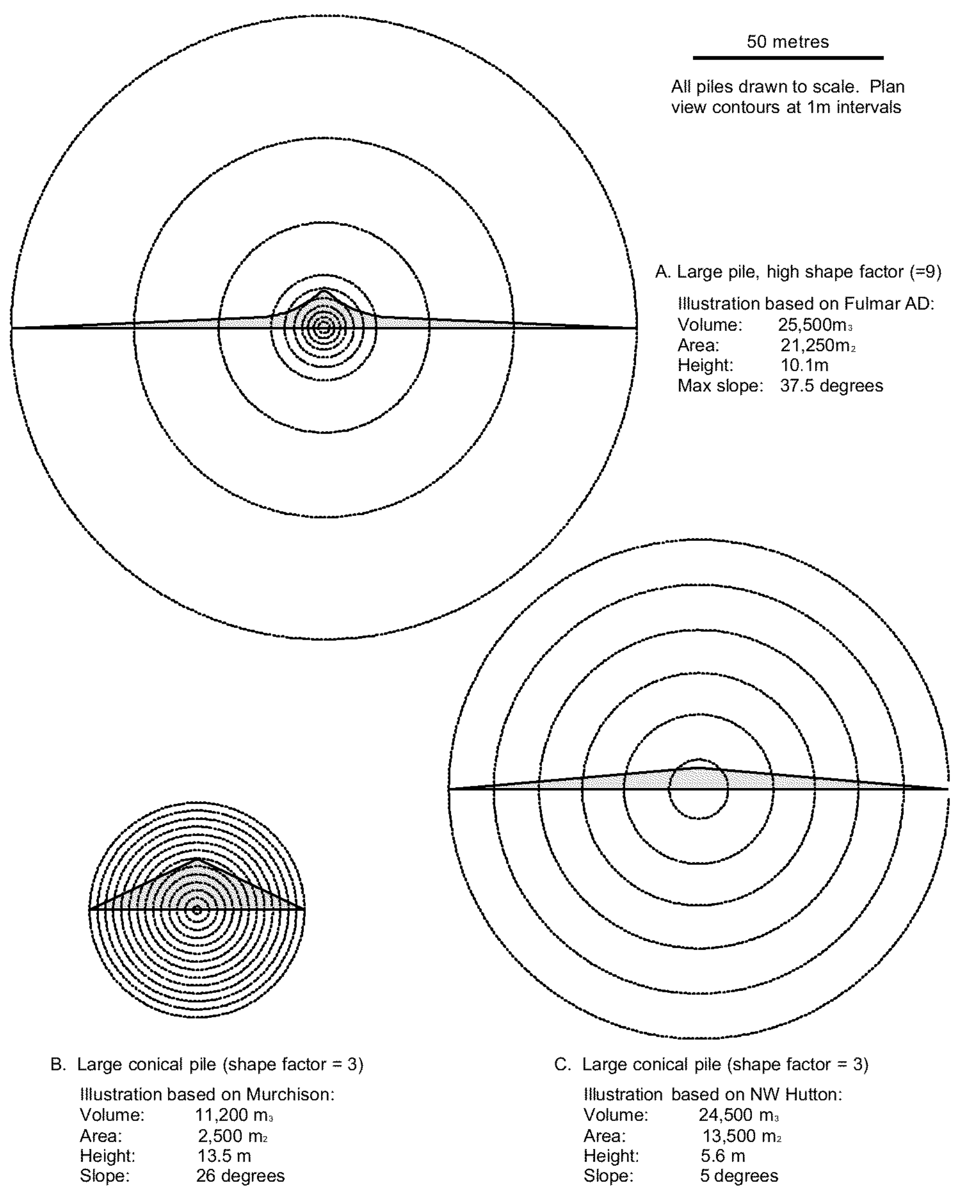


Figure 2.12. Normalised pile morphologies – cross sections and plan views - large piles (>2,500 m^3)

The three pile types are as follows:

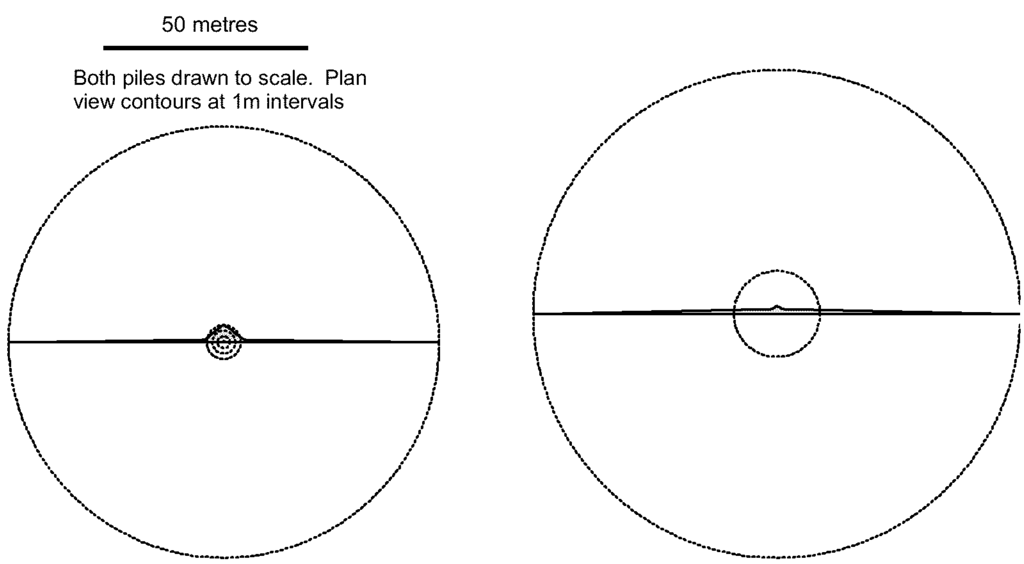
1) piles with concave slopes and high shape factors; *Figure 2.12A* shows a cross-section and plan view through such a pile based on the reported dimensions of the Fulmar pile;
2) piles with relatively uniform steep slopes (shape factor close to 3); *Figure 2.12B* shows an example based on the Murchison pile with an average slope of about 26°, this being the steepest reported uniform slope (but steeper slopes may exist at some other piles for which data are not yet available);
3) piles with relatively uniform shallow slopes (shape factor close to 3); *Figure 2.12C* shows an example based on the NW Hutton pile which has an average slope of about 5°.

Steel Structures – Small Piles

All of the small piles (which. for the purposes of this report, are defined as <2,500m³) for which sufficient data are available have shape factors greater than 5 and most are greater than 10. Two example forms are illustrated in Figure 2.13:

1) piles with extremely shallow slopes (typically less than 0.5°) with a pronounced but small mound of cuttings near the centre which may have steep (say 30-40°) slopes and be supported by a part of the structure (*Figure 2.13A*);
2) piles with very shallow slopes (typically less than 1°) with a very small mound of cuttings near the centre which may be supported by a part of the structure (*Figure 2.13B*);

In practice, it is likely that there will be no significant difference between these two types of pile in terms of cover design and construction.



A. Small pile (shape factor = 23)
Illustration based on Osprey
Volume: 1,546 m³
Area: 8,711 m²
Height: 4.1 m
Main slope: 4.5 degrees

B. Small pile (shape factor = 8)
Illustration based on Kingfisher
Volume: 1,647 m³
Area: 11,150 m²
Height: 1.2 m
Main slope: 1 degree

Figure 2.13. Normalised pile morphologies – cross sections and plan views - small piles (<2,500 m³)

Concrete Structures

There are insufficient data on the cuttings piles around concrete platforms to derive normalised morphologies. However, for reasons which are addressed later in this report, the *in situ* covering option is not considered to be viable in the case of concrete platforms.

2.5 GEOTECHNICAL CHARACTERISTICS OF CUTTINGS PILES

There are limited measured data concerning the geotechnical properties of cuttings piles. This is unfortunate because, although several investigations of cuttings piles have been undertaken, few appear to have been carried out in accordance with geotechnical engineering standards. In some cases, for example, sample and core descriptions provide little or no indication of the geotechnical properties of the materials, focusing mainly on colour and smell, rather than composition and consistency (strength).

Samples have been tested to determine water content, bulk density and particle size distribution. However, some particle size tests failed to include analysis of the fines content (*ie*. particles with effective size less than 0.063mm) and only the total combined percentage of silt and clay fractions is reported. Similarly, few samples have been tested to determine their Atterberg Limits (liquid and plastic limits) which define plasticity characteristics and provide a further insight into the likely behaviour of these deposits under changing stress/strain conditions.

The Study team recommend that future investigations of cuttings piles should be planned to maximise the amount of geotechnical data obtained. The recovery of samples is expensive and they should be described in detail by geotechnical engineers in accordance with standards such as BS5930 (BSI, 1981[1]). Adherence to appropriate testing standards, eg. BS1377 (BSI, 1999[2]), should also be required. Further recommendations, including the use of cone penetrometer tests to provide information on the *in-situ* state of the cuttings, are given in *Section 7.5*.

The following review is based mainly on data which were available during Phase I. Since the reliability of data from the various sources is variable, they are examined on a case-by-case basis in *Section 2.5.2*.

As part of this Phase II - Task 5B work, three large disturbed samples of cuttings were obtained and subjected to a range of geotechnical tests, including consolidation tests. The results are discussed in *Section 2.5.3*. Conclusions concerning the geotechnical properties of the cuttings piles are summarised in *Section 2.5.4*.

2.5.1 Components of cuttings piles

Cuttings piles comprise a wide range of materials. The principle constituents are the rock cuttings themselves. These vary in composition depending on the strata through which the wells were drilled. Even in one cuttings pile, different cuttings types can be expected ranging from shales, which are liable to decompose on exposure to water, to more resistant arenaceous rocks.

The main secondary components are mud residues and hydrocarbons. The composition of the mud residues will depend on the type of mud that was used at different levels in the well. They will also depend on the age of the cuttings piles, reflecting changes in the types of mud which have been used since exploration first commenced in the North Sea. The mud residues may include a variety of chemical and mineral additives used to enhance the performance of the mud. In addition, there may be a small proportion of metallic swarf derived from casing milling operations.

Most, if not all, cuttings piles contain varying, sometimes considerable, amounts of debris which have accumulated since the installations were first put in place. As far as we are aware, there have been no systematic, quantitative investigations of debris in cuttings piles. Anecdotal evidence suggests that the debris may include:

[1] BSI, 1981. *Code of practice for site investigations (BS 5930)*. British Standards Institution, London, 88pp.
[2] BSI, 1990. *Methods of test for soil for civil engineering purposes (BS 1377:1990 Parts 1-9)*. British Standards Institution, London.

- a large proportion of small items such as scaffold clips, welding rods, small tools, wires and ropes, sandbags;

- a smaller proportion of medium-size debris such as gas cylinders, walkways and gratings from walkways, parts of the original structure which have been cut away during modifications and maintenance; and

- occasional substantial items such as large scaffold frames, drill pipes, containers, welding sets, generators and, in at least one case, a crane.

The amount and nature of the debris will vary considerably from one installation to another but it is likely that the most debris will be found under the older installations. It is not unreasonable to expect that records will have been maintained of at least some of the larger items that have been lost but the records are likely to be incomplete, especially for older installations.

The design of *in situ* covers may be influenced by debris if it protrudes above the cuttings. Thus, a cover that includes an artificial membrane may not be suited to cuttings piles where there is protruding debris which could damage the membrane. Although it may be possible to remove the larger items before covering, thus reducing the risk of perforation, consolidation of the cuttings after covering may result in some of the remaining debris coming into contact with the membrane.

2.5.2 Previous geotechnical test data

In most cases, the previous geotechnical data are believed to have been derived from vibro-core or gravity core samples recovered from the near-surface parts of the cuttings piles. Where the cuttings materials are referred to as 'soil' in the following discussion, the term is used in its general engineering sense to mean an assemblage of solid particles with a pore fluid.

North-West Hutton (Brown & Root Survey, 1992[1])

Particle size analyses were carried out on 14 core samples taken at depths down to about 2.3m (*Figure 2.14*). The fines (<63µm) contents range from about 18% to 91% with an average of 63%. Whilst precise clay (<2µm) contents cannot be ascertained, the results suggest that this was probably less than 5% in every case.

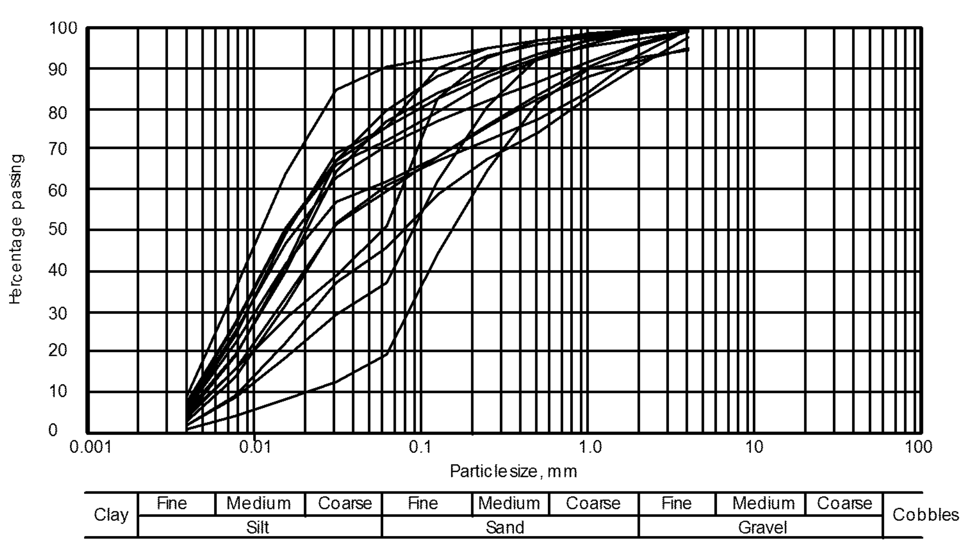


Figure 2.14. Particle size distribution of North West Hutton samples

[1] Brown & Root Survey, 1992. *North West Hutton Drill Cutting Survey – Survey Report*. Report by Brown and Root Survey Ltd, Aberdeen, to Amoco (UK) Exploration Co., November 1992.

Water contents and bulk densities show a very wide range, although it would seem that some of the specimens tested were from the underlying natural seabed soils. Excluding the latter, water contents are between 25% and 69% with an average of 52% and the bulk densities range from 1.36 to 1.89 Mg/m^3 with an average of 1.62 Mg/m^3. However, if the samples are assumed to have been saturated, or nearly saturated, the reported values are mutually incompatible, since this would imply specific gravities (particle densities) varying between 1.8 and 2.6, which seems most improbable. The water content and bulk density data are therefore unlikely to be representative of the true *in-situ* state of these soils.

Shear vane strength and compressive strength measurements are also reported for this site. A total of 15 determinations in the uppermost 2.3 m of cuttings all yielded shear strengths of less than 10 kPa. However, in view of the very low clay contents indicated by the particle size analyses it is unlikely that the shear vane test method was appropriate to the materials. For the same reason, the compressive strengths, which were all extremely low, should also be viewed with caution.

Clyde (IOE, 1998[1])

Ten core samples, probably taken near the surface of the cuttings pile, were analysed for particle size distribution (*Figure 2.15*) and bulk density. The total fines content varies between 24% and 85% with an average of 48%. Bulk densities are within a relatively narrow range of 1.68 to 1.79 Mg/m^3 with an average of 1.72 Mg/m^3.

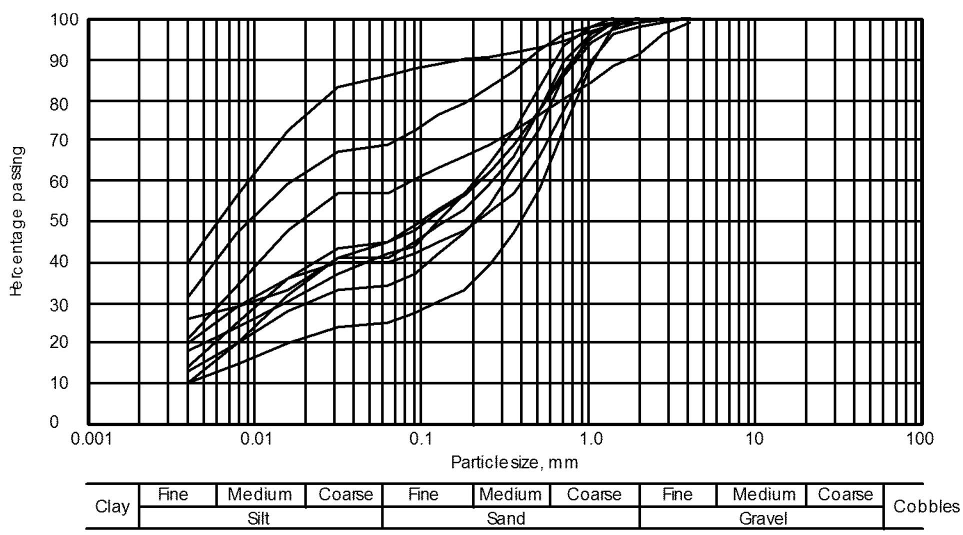


Figure 2.15. Particle size distribution of Clyde samples

Each sample was found to contain 'oil' (between 3% and 7% of the total weight) and the 'oil' and water contents were therefore measured separately. The total fluid contents, expressed as a proportion of the weight of solid particles (to correspond with the conventional geotechnical definition of water content), varied from 35% to 50%.

Heather Alpha (Grampian Surveys, 1993[2])

Eight gravity core samples were taken from the cuttings pile beneath the Heather Alpha platform and full particle size analysis was carried out on four of them (*Figure 2.16*). The total fines contents vary between 32% and 80%, of which between 2% and 22% comprise clay.

[1] IOE, 1998. *Investigation of the chemical and physical nature of the cuttings pile beneath the Clyde platform.*
[2] Grampian Soil Surveys (Aberdeen) Ltd., 1993. *Laboratory Testing – Seabed Samples, Mud Mound, Heather Alpha Platform.*

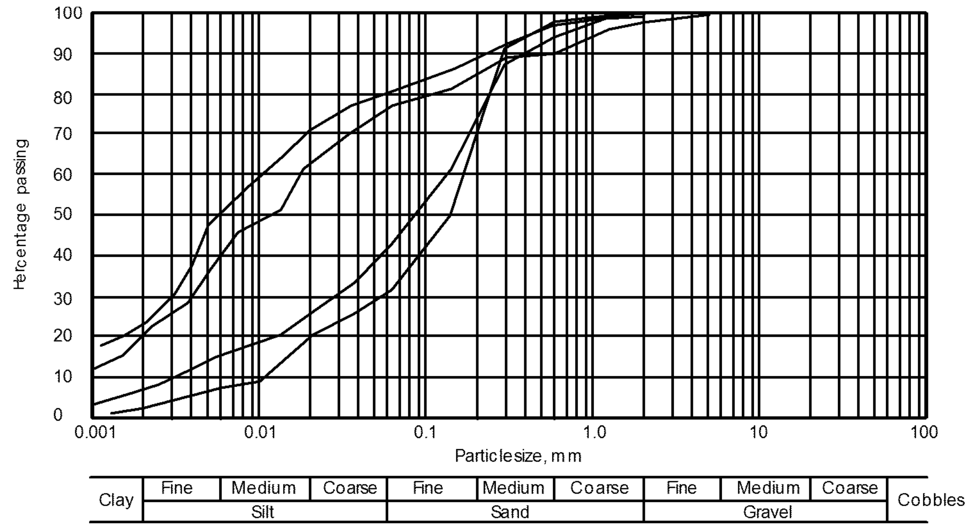


Figure 2.16. Particle size distribution of Heather Alpha samples

The Atterberg Limits show surprisingly little correlation with clay content. The plasticity indices of all eight samples are low (*Figure 2.17*) and the as-recovered water contents of six of them are substantially in excess of their liquid limits.

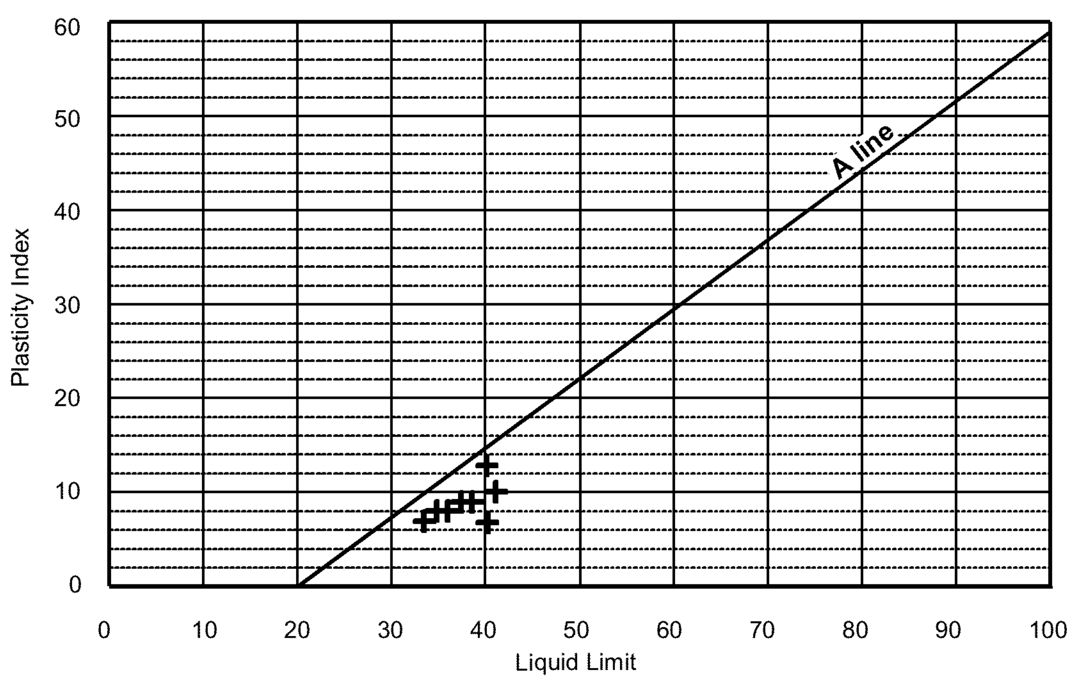


Figure 2.17. Plasticity data, Heather Alpha

The bulk densities of five samples were determined. It was noted that the remainder were too soft to test. However, the reported densities, between 1.55 and 1.83 Mg/m^3, are incompatible with the reported specific gravities (2.67 to 2.68) and water contents. There is no reason to believe that the specific gravities are incorrect but the bulk density and water content determinations may well have been affected by sample disturbance.

Beatrice (AURIS, 1992[1])

The results of tests on seven samples taken from the surface of the cuttings pile have been reported. Unfortunately, the values given under the headings 'water content' and 'density' do not appear to correspond with conventional definitions of either of these terms.

The reported 'densities', which range from 2.32 to 2.80 g/cm^3, may well be the specific gravities (or particle densities) of these soils. They are certainly outside the normal range of bulk density. Water contents, when adjusted to the conventional geotechnical definition, show a wide range, from 21% to 126%, with an average of 39%. Bulk densities derived from these water contents and (assumed) specific gravities also show large variations, between 1.35 and 2.10 Mg/m^3 with an average of 1.84 Mg/m^3. However, these correlations are based on supposition and must be regarded as tentative.

The total fines content (*Figure 2.18*) varied from 2% to 83% but no fines analyses were made to sub-divide the silt and clay fractions.

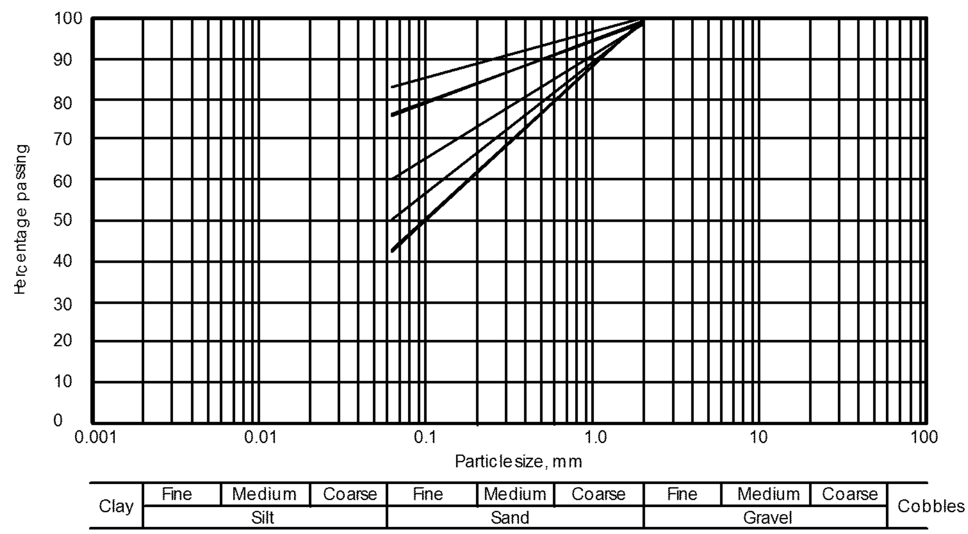


Figure 2.18. Particle size distribution of Beatrice samples

Fulmar 'A' (BGS, 1998[2])

More than 30 samples were recovered from seven sites at Fulmar 'A' at depths of between 0.9m and 5.6m. Most were tested to determine their water content and bulk density and to establish the proportions of gravel, sand and 'mud' (assumed to be fines as defined in BS1377). Shear strength measurements are also reported.

The results are incompletely presented (e.g. no units are stated for the main measured quantities) and the values given for porosity, voids and saturation are incompatible if these terms are intended to have the usual geotechnical connotation. All data from this source must therefore be treated circumspectly.

Of the 30 samples tested for particle size (*Figure 2.19*), 23 show a fines content in excess of 85%. Full particle size analysis was performed on only one sample: the 91% total fines content was found to consist of 73% clay and 18% silt. Reported water contents range between 20% and 56% with an average of 40%, and bulk densities vary from 1.49 to 1.94 Mg/m^3 with an average of 1.61 Mg/m^3.

[1] AURIS Environmental, 1992. *Environmental survey of the Beatrice field, block 11/30, 1992.*
[2] BGS, (1998). *Fulmar Alpha Drill Cuttings Mound – Moisture Content and Particle Size Analysis.* Report to Shell Research Ltd. BGS Technical Report WB/97/27C, British Geological Survey Edinburgh.

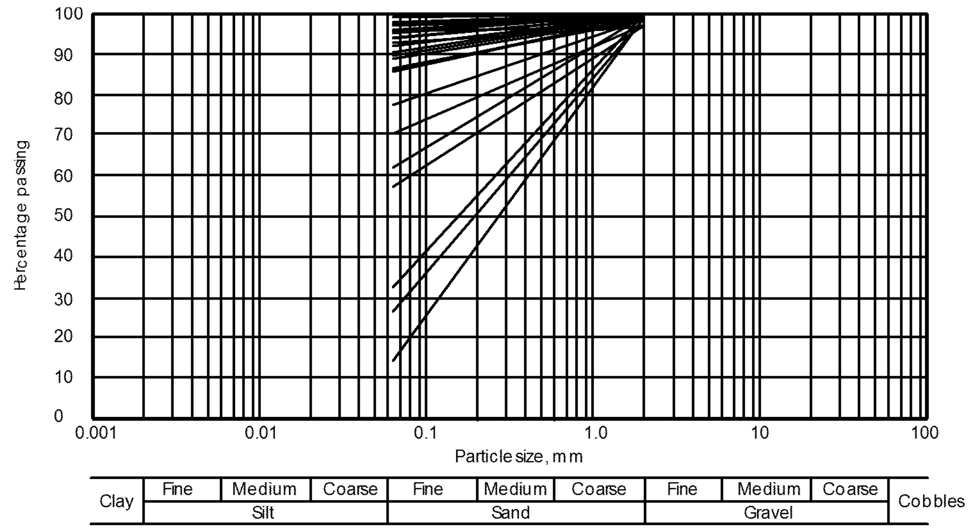


Figure 2.19. Particle size distribution of Fulmar samples

No clear depth-related trends are apparent in the values of these physical properties, nor are there any significant differences between the results from the seven sites.

A total of 42 shear strength measurements are included. They are understood to have been obtained using a small shear vane. BS1377 provides for the determination of shear strength (in terms of total stress) of soils in their sample tubes using the laboratory shear vane method. It is not clear if the strict requirements of this Standard were observed for the samples from Fulmar 'A', and is it not apparent whether any corrections for soil plasticity were applied to the measured data. It is important to note that undrained shear strengths determined by vane shear tests are likely to be valid only for uniform soft to firm cohesive soils. The presence of even small non-uniformities or coarser particles within the test zone can give rise to misleading results.

In view of the wide variations in water content and bulk density of these soils and the unknown proportions of silt and clay present in each sample, the reported shear strengths are of doubtful validity. The values, which range from 10 to 37 kN/m^2, show some tendency to increase with depth (*Figure 2.20*), but the scatter is wide. Moreover, there are no corresponding trends in the measured physical properties, such as bulk density, which appear to vary randomly. It would be unwise to conclude that these results are representative of the *in-situ* shear strengths of this cuttings pile or that they are typical of piles elsewhere.

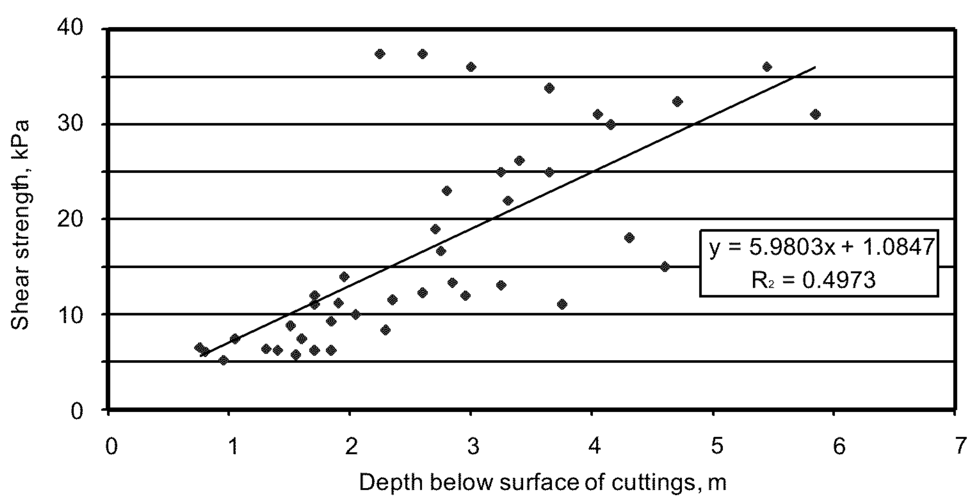


Figure 2.20. Shear strength of Fulmar samples

Beryl 'A' (Gardline Surveys, 1999[1])

Full particle size analyses were carried out on two samples from this site (*Figure 2.21*). The fines contents are relatively low, 17% and 24%, and consist almost entirely of silt with only 2% of clay particles.

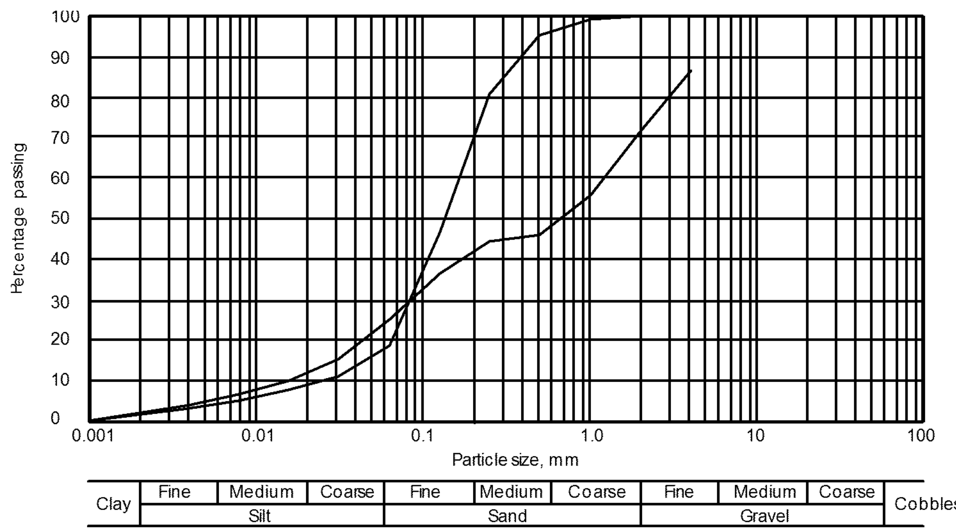


Figure 2.21. Particle size distribution of Beryl A samples

2.5.3 Task 5B geotechnical test data

Three disturbed samples of approximately 75kg were recovered in October 2000 by the study team for Phase II - Task 3; one from Beryl Alpha and two from Ekofisk. The samples are understood to have been taken from near the surface of the cuttings piles using a box corer.

Three sub-samples of approximately 25kg were delivered to the laboratories of Exploration Associates (EA) at Leamington Spa. The Beryl sub-sample was denoted S1 and those from Ekofisk were denoted S2 and S3. Tests were carried out in accordance with BS1377:1990 to determine their full particle size distributions, water contents, particle densities and Atterberg limits. In addition, remoulded specimens from each sample were tested in the largest available hydraulic consolidation cell (the 254mm diameter Rowe cell) to investigate their consolidation characteristics. Further small sub-samples were sent to the laboratories of EA's associate company, TES Bretby at Burton-on-Trent, for analysis of petroleum hydrocarbon (C8 to C37) content.

The principal physical characteristics and the hydrocarbon contents of the cuttings are summarised in *Table 2.1* (overleaf). Where two tests of the same type were carried out on specimens taken from the same sample, they are denoted A and B. There is considerable variation in properties, not only among the three original bulk samples but also between the A and B specimens from each bulk sample.

As expected, particle densities were outside the range associated with most natural soils. The A and B specimens from the Ekofisk samples gave reasonably consistent results, but with a considerable difference between the average values for the two bulk samples. However, the two specimens from the Beryl sample gave very different results: 2.79 and 3.05 Mg/m^3. These differences illustrate that particle densities will vary in accordance with the nature of the cuttings, the drilling additives and possible metal inclusions.

[1] Gardline Surveys, 1999. *Preliminary investigation of the cuttings pile at the Beryl A platform.* For Mobil North Sea Ltd.

Table 2.1 Physical characteristics of Task 5B samples

Sample No.	***S1 - Beryl***		***S2 - Ekofisk***		***S3 - Ekofisk***	
Gravel content (%)	1		7		5	
Sand content (%)	29		33		8	
Silt content (%)	54		37		62	
Clay content (%)	16		23		25	
Hydrocarbon content (mg/kg)	15,600		443		26,900	
	S1-A	***S1-B***	***S2-A***	***S2-B***	***S3-A***	***S3-B***
Particle density (Mg/m^3)	2.79	3.05	2.79	2.73	2.52	2.53
Water content (%)	31	25	71	49	50	49
Finer than 425μm (%)	100	86	86	90	80	100
Liquid limit (%)	67	32	37	38	33	70
Plastic limit (%)	35	18	17	16	19	32
Plasticity index (%)	32	14	20	22	14	18

The full particle size analyses (*Figure 2.22*) indicate total fines contents of between 60% and 87%, with the silt fraction predominant. The clay content varied between 16% and 25%. The remainder comprises mostly sand although 5% and 7% gravel were measured for the two Ekofisk samples.

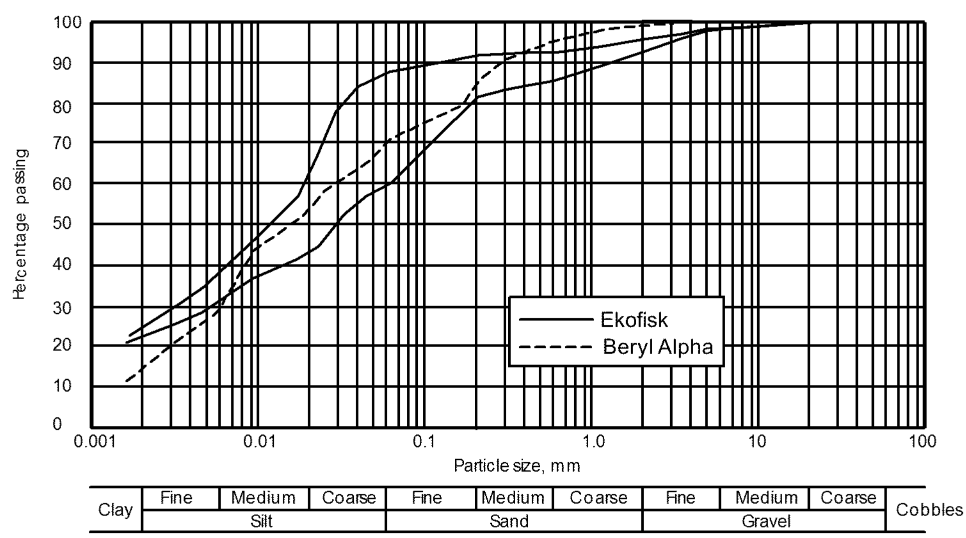


Figure 2.22. Particle size distribution of Beryl A and Ekofisk samples tested under Task 5B

Water contents are of little significance as, due to disturbance, they are unlikely to be representative of the *in situ* state of the cuttings.

Plasticity data determined from Atterberg Limit tests are plotted in *Figure 2.23*. Very different liquid and plastic limits were obtained for the A and B specimens from two of the samples (S1 from Beryl and S3 from Ekofisk). In each case, one indicates 'low plasticity' and the other 'high plasticity'. Sample S2 is of 'intermediate plasticity' but tending towards the lower values. It is noticeable that the two high plasticity specimens were taken from sub-samples having 100% finer than 425μm (ie. no particles coarser than medium sand) whereas the others had between 10% and 20% of these coarser sizes. These tests are made only on particles finer than 425μm, but this is again indicative of the erratic variation in grading from point to point.

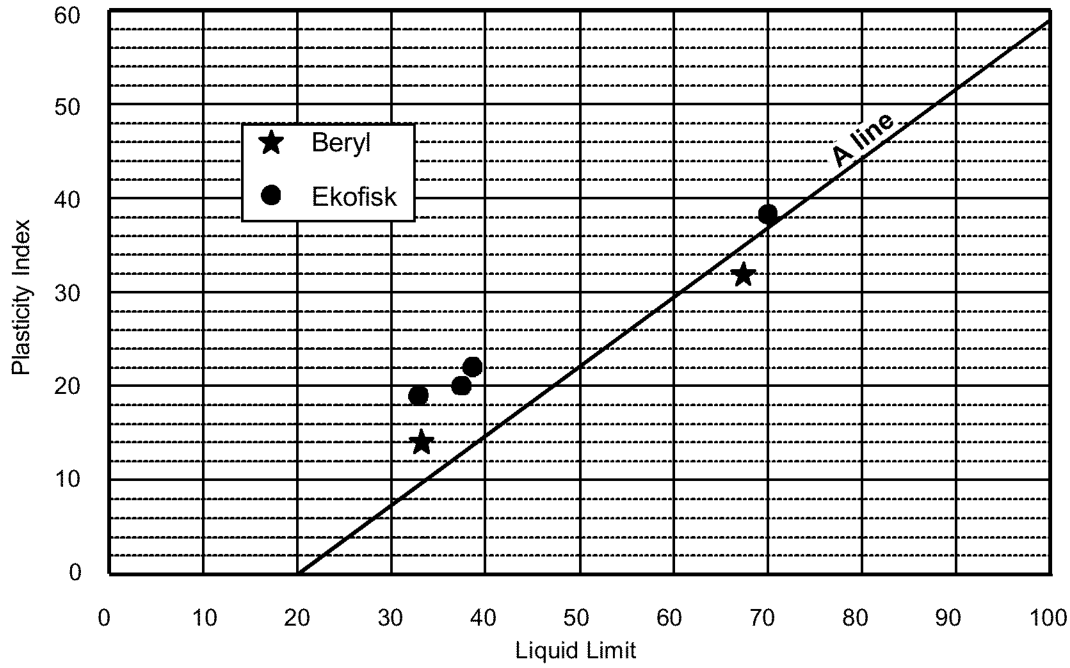


Figure 2.23 – Task 5B test data – plasticity chart

The consolidation characteristics of the cuttings determined from the hydraulic (Rowe) cell tests are summarised in *Tables 2.2, 2.3* and *2.4*. Each specimen was prepared in the 254mm diameter Rowe cell by applying a small static load to the remoulded cuttings, followed by the application of either 5 or 6 loading increments and a final unloading stage.

As the three cuttings samples were disturbed, the consolidation test results will not necessarily be representative of the behaviour of the cuttings in their undisturbed state, especially at low stress levels. However, the applied stresses were progressively increased to values consistent with the maximum *in-situ* stresses which will arise from self-weight plus the weight of the covering layers. The data are therefore considered to provide a reasonable insight into the likely behaviour of cuttings under these conditions.

Table 2.2 – Task 5B test data – hydraulic cell consolidation tests – main soil properties

Sample no.	***S1 - Beryl***	***S2 - Ekofisk***	***S3 - Ekofisk***
Specimen diameter (mm)	254.0	254.0	254.0
Initial specimen height (mm)	78.88	85.08	77.09
Particle density (Mg/m^3)	2.92 (mean)	2.76 (mean)	2.52 (mean)
Drainage condition	1-way vertical	1-way vertical	1-way vertical
Strain condition	Free	Free	Free
Initial void ratio	0.831	1.893	1.281
Final void ratio	0.575	1.103	0.914
Initial water content (%)	28.8	64.2	50.0
Final water content (%)	16.9	37.3	35.2
Initial bulk density (Mg/m^3)	2.05	1.57	1.66
Final bulk density (Mg/m^3)	2.39	1.74	1.62

Table 2.3 – Task 5B test data, hydraulic cell consolidation tests, coefficients of volume compressibility, m_v

Stress range (kN/m2)	***Coefficients of volume compressibility, m_v, m^2/MN***		
	S1 -Beryl	***S2 -Ekofisk***	***S3 - Ekofisk***
0 – 10	0.23	8.07	3.28
10 – 20	5.60	9.70	1.74
20 – 40	2.35	2.438	2.28
40 – 80	1.06	0.96	1.26
80 – 120	0.42	-	-
120 – 160	0.47	-	-
80 –160	-	0.73	0.74
160 – 0	0.25	0.11	0.34

Table 2.4. Task 5B test data – hydraulic cell consolidation tests – coefficients of consolidation, c_v

Stress range (kN/m^2)	***coefficients of consolidation, c_v, m^2/y***					
	S1 -Beryl		***S2 -Ekofisk***		***S3 - Ekofisk***	
	log t_{50}	***root t_{90}***	***log t_{50}***	***root t_{90}***	***log t_{50}***	***root t_{90}***
0 – 10	49.94	49.01	26.30	36.29	42.42	57.84
10 – 20	2.04	7.70	8.90	7.92	19.11	8.69
20 – 40	2.57	6.04	3.72	5.61	2.19	2.31
40 – 80	0.69	1.69	0.54	1.42	0.50	0.99
80 – 120	0.86	1.28	-	-	-	-
120 – 160	0.50	0.92	-	-	-	-
80 –160	-	-	7.99	7.66	0.39	0.41
160 – 0	3.16	6.33	0.26	2.82	1.51	1.99

The coefficients of volume compressibility, m_v, for each specimen are shown plotted against the applied stress during each stage in *Figure 2.24*. The initial water contents of the samples as received were very different, the Beryl sample being much drier (28%) than those from Ekofisk (64% and 50%). This, allied to its greater average particle density, as mentioned above, meant that when prepared under a small static load, its initial density was much higher than the others – about 2.0 Mg/m^3 compared with about 1.6 Mg/m^3 for S2 and S3.

The effect of this initial difference in physical properties is apparent in the coefficients of volume compressibility, m_v, obtained from the first loading stage, from 0 to 10 kN/m^2, for which Beryl-S1 exhibited substantially lower compressibility than the Ekofisk samples, in effect behaving as a pre-consolidated specimen. However, by the third loading stage, from 20 to 40 kN/m^2, it is apparent that the responses of the three samples were similar and the measured m_v values are high. Thereafter, the values decrease with increased stress but, overall, all of the cuttings samples are highly compressible throughout the range of stresses likely to apply in practice.

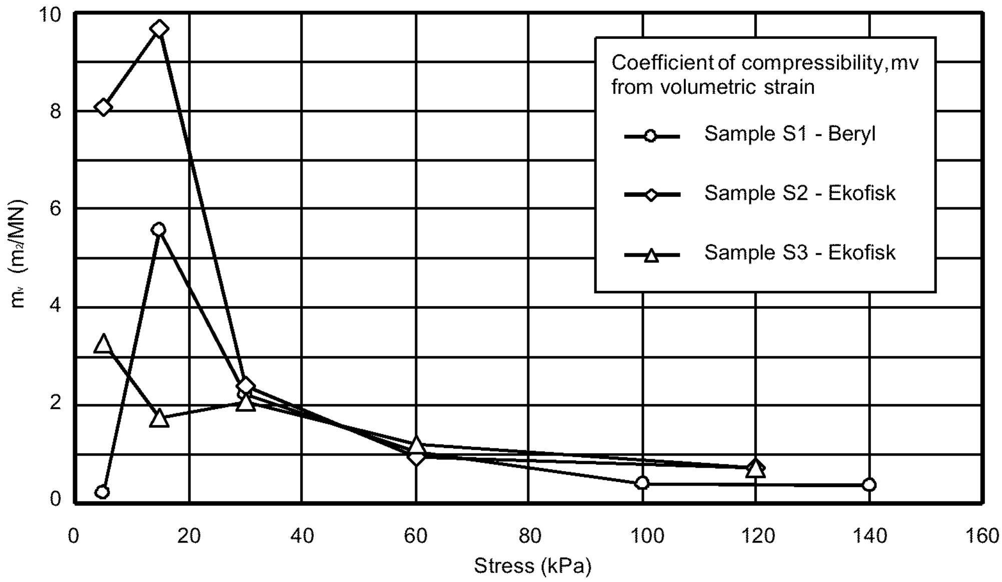


Figure 2.24. Hydraulic cell consolidation tests – coefficients of volume compressibility

Graphs of void ratio against the logarithm of applied stress for each test are shown in *Figures 2.25, 2.26* and *2.27* (the first stage loading from 0 to 10 kN/m^2 is not shown). For most natural normally-consolidated soils, the curve during compression would be expected to be approximately linear. Bearing in mind the heterogeneous nature of the cuttings samples, this is reasonably so for the samples tested, especially if the first two loading increments up to 20 kN/m^2 are disregarded.

For stresses between 20 and 160 kN/m^2, the slopes of these curves can be used to define the compression index, C_c, of each sample and are thus indicative of their relative compressibility. It can seen that Ekofisk-S2 was the most compressible, particularly during the early stages, and Beryl-S1 the least. The average compression indices for S1, S2 and S3 are approximately 0.23, 0.37 and 0.35, respectively.

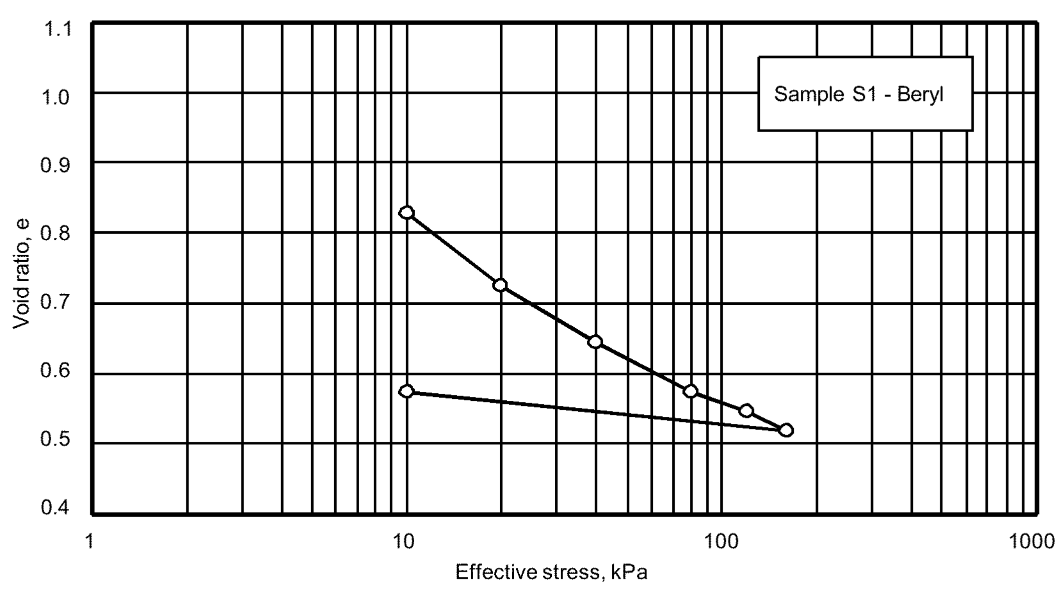


Figure 2.25. Hydraulic cell consolidation tests – void ratio v. effective stress –Beryl-S1

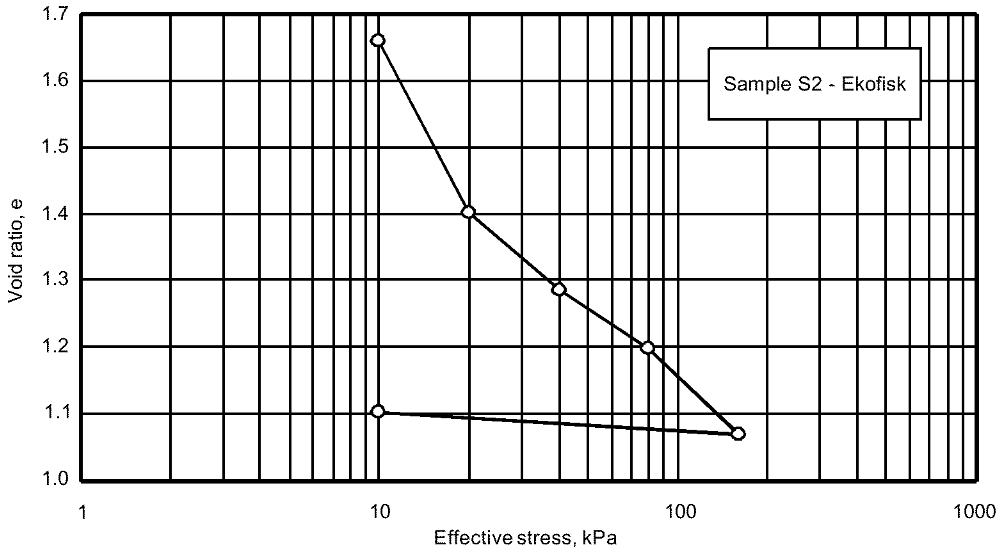


Figure 2.26. Hydraulic cell consolidation tests – void ratio v. effective stress – Ekofisk-S2

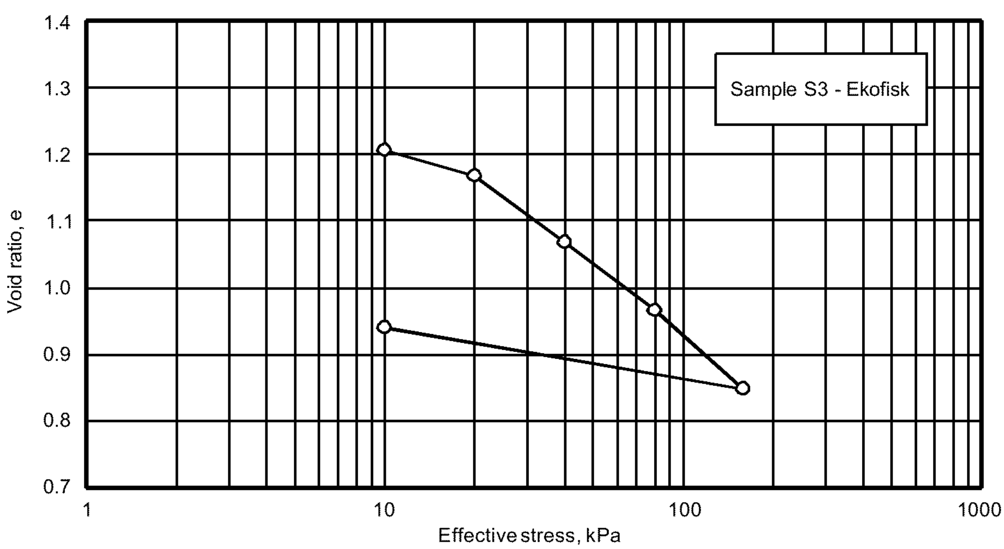


Figure 2.27. Hydraulic cell consolidation tests – void ratio v. effective stress – Ekofisk-S3

Much greater variability was observed in the coefficient of consolidation, c_v which is the parameter governing the rate of dissipation of pore water pressure and hence the rate of consolidation of the cuttings under applied loading. Moreover, there are a number of anomalies in the test results which, if obtained from tests on natural soils, might call into question some of the data obtained. However, it seems very possible that this abnormal behaviour could be associated with the variable nature of the constituent particles and also the variable content of the pore fluid. Conventional analytical techniques, which are based on idealised natural soils, are unlikely to be so relevant to these cuttings.

Several methods of analysis are available for deriving c_v values from hydraulic cell consolidation tests. For free-strain conditions, the method based on volumetric strain (ie. overall change in volume with time) for each applied load is usually the most suitable. The results listed in *Table 2.4* and plotted against applied stress in *Figures 2.28, 2.29* and *2.30* below, are based on this method and show two values ('log t_{50}' and 'root t_{90}') derived from the test data. The very high values shown in the table for the first stage loading to 10 kN/m^2 have been omitted from the graphs for clarity. Since c_v is related to soil permeability, the greater the value of c_v the greater will be the rate of dissipation of pore pressure and hence (for other conditions equal) the greater will be the rate of consolidation of the cuttings.

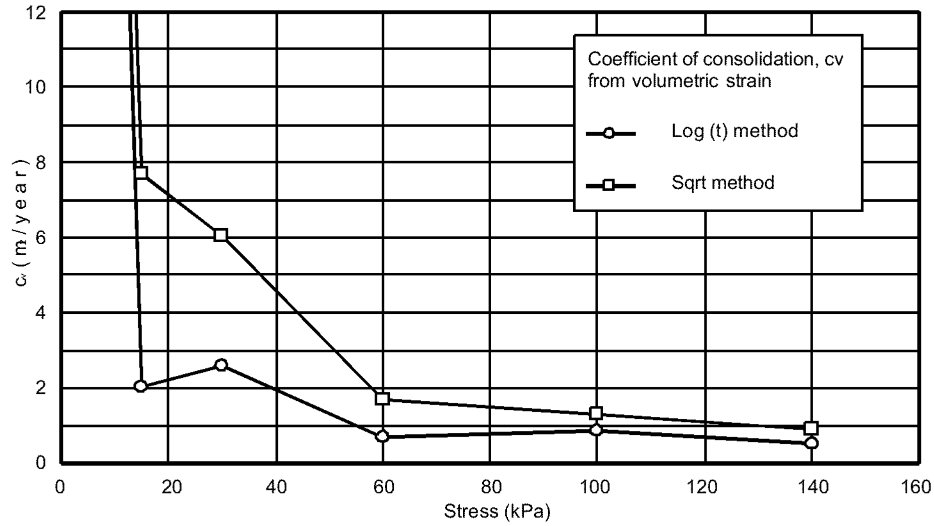


Figure 2.28. Hydraulic cell consolidation tests – coefficients of consolidation – Beryl-S1

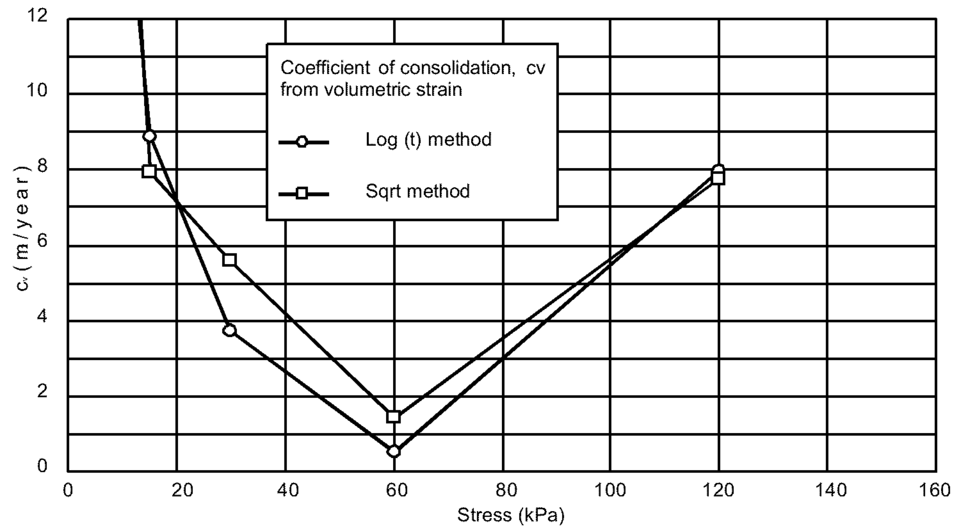


Figure 2.29. Hydraulic cell consolidation tests – coefficients of consolidation – Ekofisk-S2

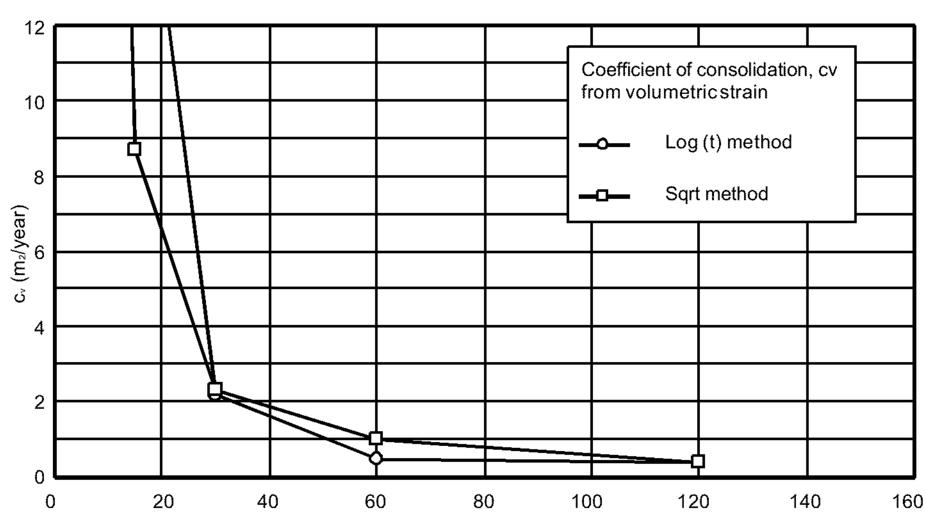


Figure 2.30. Hydraulic cell consolidation tests – coefficients of consolidation – Ekofisk-S3

The consolidation test specimens were prepared simply by introducing the as-received disturbed cuttings samples into the Rowe-cell and applying a small static load. Since the samples were initially loose, during the first loading stage (to 10 kN/m^2) they exhibited high c_v values of between 26 and 58 m^2/y. To put this into context, for a c_v of say 40 m^2/y, a 10m layer of cuttings loaded to 10 kN/m^2 would require about 2 years to reach a 90% degree of consolidation.

During subsequent loading stages, the c_v values obtained generally decrease sharply with increased stress as the particles are forced closer together and the void ratio decreases. Between 40 kN/m^2 and the maximum of 160 kN/m^2, values are generally less than 1.0 m^2/y. Again, by way of illustration, a 10m layer of cuttings in this condition would now require 85 years to achieve 90% consolidation, much longer than the age of the existing cuttings piles.

However, there are a number of anomalies in these results, the most significant of which is the sudden increase in c_v for Ekofisk-S2 during the final loading stage from 80 to 160 kN/m^2. Having exhibited progressively decreasing coefficients with increasing stress, and with an average value of about 1.0 m^2/y for the previous stage from 40 to 80 kN/m^2, the apparent coefficients for final stage seem to have increased to about 8 times this value, suggesting a more permeable soil structure despite a decrease in void ratio. This is uncharacteristic of normal soils. The reasons for this anomaly are unclear. Since the results for Beryl-S1 and Ekofisk-S3 during final stage loading (about 0.7 m^2/y and 0.4 m^2/y, respectively) certainly do not reflect this trend, it would be prudent to ignore the higher Ekofisk-S2 values for this stage.

In view of the relatively large silt and sand contents of the samples, the c_v values obtained from these tests are surprisingly low, especially at higher stress levels. On this basis, except for cuttings heights of only a few metres, consolidation due to self-weight will be far from complete and excess pore pressures within the piles can be expected to remain high.

2.5.4 Summary of geotechnical characteristics

Despite the uncertain reliability of much of the previous test data, some general conclusions may be drawn about the geotechnical characteristics of the cuttings piles at the various sites from which samples were taken. The recent series of tests was carried out strictly in accordance with BS1377:1990 and the data obtained are considered more reliable but it should be noted that the all of these tests were performed on disturbed samples. The test results do not therefore provide direct evidence of the important *in-situ* state of the existing cuttings at these locations. However, it is recognised that obtaining undisturbed samples will be very difficult and costly and that a combination of *in situ* testing (see *Section 7.5*) and detailed laboratory testing on reconstituted samples may be the only practical way of increasing our knowledge of the geotechnical characteristics of cuttings piles.

The range of physical properties measured for samples of the cuttings piles is very wide. This is not surprising in view of the known variability of their constituents and the manner of their deposition. Bearing in mind the small amount of available data, it is unlikely that the data encompass the full range of properties of the cuttings piles for which covers might be proposed.

Particle Size

Very few test results have shown more than about 3% of gravel-size particles (2mm-60mm). Two exceptions, both close to 10%, were noted in the Beatrice data. Whether the gravel fraction consisted of particles marginally above 2mm or much coarser particles is unknown but it appears that the gravel content is insignificant and that the cuttings piles consist of varying proportions of sand (0.06mm-2mm), silt (0.002mm-0.06mm) and clay (<0.002mm).

With the exception of the Fulmar 'A' samples, the sand content varies between about 10% and 80% with an average roughly mid-way between these extremes. At Fulmar 'A', most samples were found to have less than 10% sand but some had a much greater proportion, in one case 85%. Where full particle size analyses have been carried out, the measured clay content was from about 2% to 25%, but indications from the other tests, particularly those at Fulmar 'A', suggest that the full range may extend to 30% or 40% or more. At some sites, such as Clyde and Heather Alpha, clay contents seem very variable, whereas at others, notably NW Hutton, they appear consistently low.

Density, Water Content and Specific Gravity

Reliable density measurements require undisturbed samples. Water contents can be determined from disturbed samples provided that they are properly sealed to prevent moisture loss. Specific gravity measurements are the least demanding in terms of sample quality, but it is clearly essential that the test specimens cover the full range of particle types present. Given the wide variety of constituents, a small number of tests is unlikely to indicate the full range of specific gravity of the cuttings. The mutual incompatibility of many of the earlier data sets makes it difficult to assess which of the measured properties are more likely to indicate *in-situ* conditions.

Excluding one extreme and unlikely value from the Beatrice site, reported water contents range between 18% and 69%. In some cases, such as Clyde, values fall within a relatively narrow band (27% to 44%); in others, the range is much wider. A similar wide range is evident in the reported bulk densities which vary between 1.36 to 1.94 Mg/m^3. Values outside this range can be inferred from other test data at some sites but this is speculative.

It seems likely that its variability is at least partly a reflection of the variable nature of cuttings constituents. Most of the results, including those taken at up to 5m depth within cuttings piles, indicate a soft or loose condition which is what might reasonably be expected of deposits formed in this manner. It is unclear whether the higher densities and lower water contents are a genuine reflection of *in-situ* conditions or the result of sample disturbance. For present purposes, it would be prudent to assume the latter.

In only one of the earlier test reports are the particle specific gravities listed; four tests on samples from Heather Alpha gave 2.67 or 2.68, which are typical of natural soil minerals. The 'densities' given for Beatrice are probably intended to be specific gravities. These show a much greater variation, between 2.32 and 2.80. The results from the recent tests on the Beryl and Ekofisk samples, 2.52 to 3.05, suggest that the range is wider than that of natural soils.

One further aspect of the physical properties of the cuttings is the nature of the pore fluid in the 10 core samples from the Clyde site. All contained 'oil' – between 3% and 7%, expressed as a proportion of the total soil weight, or 13% to 19% by weight of the pore fluid. There is some tendency for oil content to decrease with increasing fines content but this may not be significant. Results of the recent analyses for this Study indicated hydrocarbon contents of about 1.6% and 2.7% for the Beryl sample and one of the Ekofisk samples, whereas the result for the second Ekofisk sample was negligible. However, the distribution of hydrocarbons (and of other contaminants) in the pore fluid is unlikely to be uniform.

Shear Strength

It has been reported that the upper parts of some cuttings piles consist of a very soft, mobile layer with a gelatinous consistency through which divers can wade. In contrast, there are also reports that the surface of some piles comprises a hard crust or has the consistency of plasticene.

Anecdotal evidence suggests that, below the surface layer, strength profiles may be very irregular and that there is no basis for assuming that well-developed strength gradients will be encountered in cuttings piles. Almost all those with whom we have discussed pile properties, and who have direct experience of working with cuttings during maintenance and construction work, have stressed the variability of cuttings consistency both within single piles and from one pile to another.

The only previous test data which purport to indicate shear strengths of the cuttings piles are of doubtful validity, as discussed in *Section 2.5.2*, and are unlikely to be representative of their *in-situ* strengths. During discussions with promoters of cuttings recovery systems in Phase I of the JIP, several indicated that they had encountered cuttings with shear strengths in excess of 100 kPa. However, such high strengths are thought likely to be unusual.

A large sample of cuttings delivered to shore for treatment was examined in November 1999 by a member of this Research Team. While not necessarily representative of the material in cuttings piles, the material was observed to be markedly thixotropic.

Consolidation

The only data concerning the consolidation characteristics of cuttings piles are those obtained during this Study. The samples were found to be highly compressible and to exhibit slow rates of consolidation, especially at the higher stress levels compatible with the maximum stresses likely to arise from self-weight and the applied covers.

The slow consolidation rates are somewhat surprising in view of the relatively low clay contents of the samples. However, the composition of both the cuttings particles and the pore fluid clearly differ from those of natural soils. This may also explain the erratic compression-time behaviour observed during some of the loading stages. Whilst the measured hydrocarbon content of the pore fluid was relatively low, the effect of drilling additives and other contaminants may well be responsible for these deviations. For present purposes, it is assumed that the consolidation properties obtained from these tests are representative of the *in-situ* behaviour of the cuttings.

2.5.5 Late additional data

As this report was being finalised, the results of additional testing undertaken at Reading University were distributed. These comprised two Rowe Cell tests on samples from Beryl and Ekofisk, together with supporting data on index properties and pore water chemistry. The data were received too late to be fully incorporated in the analyses presented in this report but it is noted that the results differ in some respects to those presented above. In particular, higher values of c_v were observed at high stress levels. These may be a reflection of variation of cuttings properties (even within individual piles), different test procedures or equipment design. It is not possible, without undertaking additional tests, to provide a positive explanation for the differences. However, it is noted that, if the c_v of the Beryl and Ekofisk cuttings is indeed higher than the values used in this assessment:

- some cuttings piles may have consolidated under self-weight to a greater extent than is assumed here;
- as a result, such piles would be more stable, and
- rates of pore water expulsion under the additional load of the cover may be greater.

These comments are reiterated where appropriate in Sections 7 and 8 below. Reading University's report on the additional tests is reproduced as Appendix A to this report

3 COVER DESIGN CONSIDERATIONS

3.1 GENERAL REQUIREMENTS

The first stage of design is to identify the functional requirements of the cover. This is entirely dependent on environmental objectives, *ie.* the extent to which it is necessary to isolate the cuttings from the marine environment. There are two components of isolation which need to be considered:

1) Chemical isolation - prevention (or reduction to an acceptable rate) of the chemical migration of contaminants through the cover material and/or the prevention of release of contaminated pore water expelled from the cuttings pile during consolidation;
2) Biological isolation - prevention of direct access to the contaminated materials by organisms.

The cover must continue to fulfil its function for a long period of time. The performance specification must therefore consider possible time-dependent degradation of the cover and reduction of its effectiveness. This may give rise to a requirement for a monitoring and maintenance programme.

The second stage is to identify the materials (or combinations of materials) which meet the requirements concerning effectiveness and durability and to establish whether or not pile covers can be constructed with those materials using existing technology.

The third design stage is to modify the design in order to minimise any potential adverse impacts which arise due to the physical presence of the covered pile on the seabed. These include the potential for damage to vessels or fishing gear. This stage of the design process should also examine the potential for modifying the cover in order to enhance habitat or to provide new beneficial habitat.

In evaluating the overall viability of covering as an option, it is necessary also to take into consideration the works that needs to be undertaken during decommissioning of the structure and the requirement for future maintenance and monitoring. The maintenance burden might be reduced by design modifications.

3.1 PERFORMANCE REQUIREMENTS

The purpose of the cover is to ensure that the contaminants which are contained in the cuttings piles are prevented from escaping to the marine environment or that the rate of loss, if any, is below that which may be expected to give rise to unacceptable environmental impacts. The cover must be designed to provide the required degree of effectiveness in the long term. The definition of long-term should recognise that, if a cover is designed to permit a slow (ie. acceptable) release of contaminants, a stage may be reached when degradation of the cover may have little or no adverse effect because the nature or degree of contamination within the cuttings pile has changed. There are five main considerations:

1) release of loss of contaminants (ie. chemical isolation);
2) biological isolation;
3) resistance to tidal and wave-induced currents;
4) resistance to trawling and anchoring;
5) resistance to structural collapse.

3.1.1 Release of contaminants (Chemical Isolation)

There are two approaches which can be adopted with respect to contaminant loss:

1) prevent for all time any contaminants release;
2) permit release of contaminants at a rate which does not give rise to adverse environmental impacts.

Depending on the types of covering materials which are used it might, in theory, be possible to prevent any loss of contaminants through the cover. However, if there is no release, and assuming that the contaminants do not degrade to less harmful substances, this approach ensures that the cuttings piles will not be given the opportunity to improve over time and implies that:

- either the design must be sufficiently robust to resist, over geological timescales, any conceivable impacts which may result in a reduction of effectiveness, or
- at some stage in the future, technology will be available to recover and deal with the covered cuttings piles, ie. covering is an interim solution.

The approach adopted in this Study has been to assume that, at present, there exists no method by which the cuttings piles can be wholly isolated, without any loss of contaminants, over geological timescales. The rationale behind this decision is explained in *Section 4* which deals with construction materials and methods. The Study team has therefore sought to identify the rate of contaminant loss which will not give rise to unacceptable impacts and to develop, with reasonable margins of safety, cover designs which ensure that such rates will not be exceeded. Contaminant loss rates and their potential impacts are considered in *Sections 7* and *8* respectively.

3.1.2 Biological isolation

Bioturbation is an important consideration because the thickness of the cover which is required to biologically isolate deposited material is typically greater than that needed to achieve chemical isolation (Brannon *et al* 1985)[1]. Biological activity within the completed cover may cause contaminated material to escape from the contained cuttings by the following routes:

- accelerated diffusion through vertical tubes created by infauna and the increase in water movement caused by respiration or feeding by sediment-dwelling animals, and
- uptake of contamination by infauna due to ingestion or absorption and subsequent biomagnification.

Burrowing organisms colonise the newly constructed cover by settlement of planktonic larvae and immigration from surrounding areas. Therefore burrowing organisms need to be prevented from reaching the contaminated materials or the lower parts of the cap which may become contaminated over time due to chemical and pore water migration.

Species that could potentially penetrate the cover material are infaunal burrowers, in particular decapod crustacea. A species of burrowing shrimp, belonging to the genus *Axius*, has been recorded burrowing to depths of 3 metres off *Nova Scotia* (Pemberton *et al* 1976)[2] . Such deep burrowers are likely to inhabit muddy sediments and would not populate a cover constructed of coarse materials.

It is speculated that, depending on the precise characteristics of the substrata exposed to the water column, the fauna which colonise the cover will be dominated by filter feeding and mobile predatory species. Apart from the surface 3 to 5cm where deposition of material may occur most potential cover materials will be very low in organic carbon therefore the infaunal biomass will be correspondingly low. In addition species likely to colonise gravel would be unlikely to migrate vertically into a lower sandy habitat.

3.1.2 Resistance to tidal and wave-induced currents

The materials which are used to construct the covers must be able to resist the action of wave and tide-induced water currents. Analyses of relevant wave and current data are presented in *Section 5* and armouring requirements are considered in *Section 6*. These data are also relevant to methods and costs of construction.

[1] Brannon J.M., Hoeppel R. E., Sturgis T.C., Smith I., jnr and Gunnison D. 1985. *Effectiveness of capping in isolating contaminated dredged material from biota and the overlying water*. US Army Corps of Engineers Waterways Experimental Station, Technical Report D-85-10.
[2] Pemberton, G., S., Risk M., J., and Buckley D., E., 1976 *Supershrimp. Deep bioturbation in the Straits of Canso, Nova Scotia*. Science 192 790-791.

3.1.3 Resistance to trawling and anchoring

The cover must be able to resist the impacts of trawling. This criterion applies only to piles where the structure has been removed down to the level of the cuttings pile or where the pile extends well beyond the structure footprint and the pile can therefore be deliberately or accidentally trawled. In those cases where part of the structure remains protruding above the cuttings pile, consideration will need to be given to the risk that the remaining structure presents to fishing vessels and personnel.

If the structure is removed sufficiently to permit trawling over the pile, the cover must be designed not only to resist damage but also to ensure that the potential for damage to fishing gear (and the consequential risks to fishing vessels and personnel) is minimised.

3.1.4 Resistance to structural collapse

In some instances, a part of the installation may be left in place. In time, the remaining structure will decay and collapse. Collapse may be progressive, with relatively small parts of the structure falling away or it may be catastrophic, involving the collapse of a major part of the structure. Clearly, the cover must be able to resist the impacts of structural collapse.

Collapse scenarios are also relevant to the timing of a covering operation. It may, for example, be necessary to install a cover prior to any removal of the structure or part of the structure in order to minimise the risk of unacceptable contaminant release arising from pile disturbance during decommissioning. Such disturbance may occur due to accidental release of parts of the structure during decommissioning or due to the effects of the plant and methods used to remove part or all of the structure. In this context, a two-stage covering solution might be considered, the first stage comprising the placement of an initial protective layer of material to minimise the impacts of accidental disturbance during decommissioning followed by the placement of a more permanent and durable layer of material when the installation has been removed.

3.2 CONSTRUCTION MATERIALS AND METHODS

The design of a cover must incorporate materials with the required isolating properties and which are sufficiently durable to resist the impacts described in the preceding section. Potential construction materials must, however, also be reviewed in terms of constructability. There is little point, for example, in designing a cover which incorporates wholly impermeable materials if they cannot be assembled using existing technology to provide a cover which, as a whole, is impermeable. Potential materials and methods of construction are reviewed in *Section 4*.

3.3 ECOLOGICAL ASPECTS

Having identified the performance requirements of the cover and the design options that can meet those requirements, it is then necessary to review the final form of the cover. While this may have little or no bearing on its performance, it is relevant to its potential for adverse ecological impacts or, conversely, its potential to increase habitat diversity;

Cuttings piles that are covered *in situ* will inevitably form mounds but the size of the mound and its final shape can be designed to enhance its value as a reef, *ie.* it could be made much larger than is required for containment purposes and its form and surface character could be optimised to provide a range of habitats. The cap could be designed in such a manner as to present little or no obstacle to trawling gear, but increase habitat diversity. These aspects are considered in *Section 9*.

4 POTENTIAL MATERIALS AND METHODS OF CONSTRUCTION

4.1 INTRODUCTION

The materials which might be used to construct pile covers need to be evaluated in terms of:

1) suitability in relation to the required function, including long-term durability;
2) practical aspects of placement (ie. constructability);
3) cost of provision and placement;
4) energy budget for construction.

This section presents an initial screening of options based principally on an assessment of functionality and constructability. The cost of construction is also considered at preliminary level but is not used as the basis for the elimination of any material options. Energy budgets for construction are not included in this initial screening on the basis that the differences of energy consumption are most unlikely to be sufficiently great to warrant exclusion of any material options. Energy budgets are provided in *Section 11* only for those options carried forward to the final assessment. The basis of the initial screening for functionality, constructability and cost are summarised below.

Cover construction materials may fulfil one or more of the following functions:

1) prevent loss of contaminated solids;
2) prevent loss of contaminated fluids expelled from cuttings;
3) prevent direct access to the cuttings by organisms;
4) protect other materials used in cover construction;
5) provide a suitable substrate onto which other functional materials are placed;
6) provide an external surface designed, for example, to enhance habitat or reduce risks to others (eg, risk to trawling).

In addition, the cover should serve to enhance long-term pile stability. Potential construction materials are reviewed to identify the functions which they could fulfil and their efficacy compared with other materials.

Construction methods are only briefly reviewed in *Sections 4.2* and *4.3* in the context of each of the potential construction materials. Attention is focused on identifying whether or not proven placement technologies exist which can be used to place the materials and on the overall ease of construction. Materials for which there are no proven placement technologies or which could be placed but in such a manner that their intended functionality is likely to be significantly downgraded, are eliminated from further consideration. A more detailed description of construction methods is provide in *Section 4.5* only for those materials that are considered suitable and practical for use in pile covers.

4.2 NATURAL CONSTRUCTION MATERIALS

4.2.1 Cohesive soils

Cohesive soils (clays and clayey silts) have a very low permeability and could therefore be considered for the prevention of solids loss and to minimise the rate of fluid release. If they could be placed at a sufficiently high density, they would be resistant to erosion by currents but their resistance would likely reduce over time. They would have limited ability to withstand trawling impacts if placed at a high density but this too would reduce with time and as the number of impacts rises. They have no ability to withstand the impacts of anchoring or structural collapse unless placed in excessively thick layers.

Cohesive materials would be extremely difficult to place without causing significant disturbance to the cuttings. They have no advantage over sand in terms of their ability to prevent solids loss and would only merit further consideration if it were shown that fluid contaminant loss is a critical issue. Their use appears to have little merit with the possible exception of their use as contaminant absorbents in mattresses (*Section 4.3.5* below).

4.2.2 Peat

Peat could be used to prevent or limit solids release but cannot prevent fluid release. It would have no resistance to any external impacts and would be impossible to place in an uncontained state because of it's very low particle density.

Use of peat, irrespective of difficulties of construction appears to have no merit with the possible exception of its use, if contained in geotextiles, as a contaminant absorbent (*Section 4.3.5* below).

4.2.3 Granular (non-cohesive) soils

Granular soils comprise sand, gravel and stone (ie. cobbles and boulders). These materials are widely used in offshore construction and there are several proven methods of placement. Existing applications include:

- protection of pipelines against anchors;
- protection of pipelines against bottom trawled fishing gear;
- ensuring lateral stability of pipelines;
- ensuring vertical stability of pipelines (upheaval buckling);
- insulation of hot pipelines;
- scour protection at platforms;
- platform ballasting;
- special applications to cover seabed obstacles such as pipeline and cable crossings, tees, valve assemblies and shipwrecks.

Sand

Sand could be used to prevent loss of solids. While it is permeable, and therefore cannot prevent the passage of fluids, it might be considered for use as an accommodating material to contain contaminated water expelled from the cuttings during consolidation. This was the recommendation of the JIP Phase I, Task 5.1 report. The general principle is that contaminated leachate expelled from the cuttings will be taken up by the sand (if placed in a sufficiently thick layer). The leachate may, in the long-term, leak from the sand layer but at a slow rate. The sand layer provides a suitable substrate for the placement of heavier materials that protect the cuttings from erosion by currents and from impacts such as trawling and anchoring. A particular advantage of sand is that it is the only material that can be placed directly on top of the cuttings with minimal disturbance, provided the additional weight does not give rise to shear failure.

Gravel

Gravel might be used as the first stage of a protective layer for an absorbent sand layer or for other materials. Gravel could also be used to modify the external surface of a cover with heavy armour stone in order to provide a smooth surface for the passage of trawl boards. This approach is used in the Dutch Sector where <80 mm material is sprinkled over coarse armour. Larger materials cannot be used for this purpose as they would tend to be trapped in fishing nets.

Stone (ie. cobble and boulder size rock)

Cobbles (60 – 200 mm) and boulders (>200 mm) are ineffective at retaining either solids or fluids. However, they can be used to provide an armour layer over other materials which, if the stone is sufficiently large and/or it is laid in sufficient thickness, will be highly resistant. Stone can be placed using fall pipe vessels or side stone-dumping vessels (*Section 4.5*).

4.3 ARTIFICIAL MATERIALS

4.3.1 Impermeable membranes

Impermeable membranes could be used to contain both solids and liquids. However, as it appears that many cuttings piles have pore pressures which are in excess of hydrostatic, the construction of an impermeable cover would greatly reduce the rate of pore pressure dissipation and hence increase the risk of instability under dynamic loads (*eg*. earthquakes, wave loading) and under the weight of additional protective materials. There are also several almost insurmountable problems with respect to installing a large area of membrane and ensuring that it is not damaged during and after installation. These are summarised below.

1) Membranes are delivered in rolls which would have to be laid in sections and joined underwater to form a wholly impermeable seal. At present, there is no proven method of forming impermeable seals underwater. Laying overlapping sheets would not ensure impermeability.
2) Special seals would be required where the membranes abut against any remaining structural members; in theory, these could be formed using purpose-built sleeves, bitumen or concrete but the difficulties of installation would be severe and it would be impossible to ensure and verify a complete seal. Long-term degradation of seals would also need to be considered.
3) Membranes are difficult to place underwater, even when there is no requirement to form perfect seals between adjacent or overlapping sheets. They would need to be weighted to counteract their inherent buoyancy and would be very difficult to manoeuvre in deep water with a current, particularly under and around any remaining structures. The deepest known deployment of membranes is the covering of the ferry Estonia in about 70 metres of water but the technical requirements in that case were different to those here.
4) Wave-induced differential pressures on the membrane could result in rupture (exacerbated by any protruding debris) and erosion at the edges.

Even if it were possible to construct an impermeable membrane, there are additional disadvantages:

1) In order to avoid damage during installation (and during placement of overlying protective materials) by debris protruding from the cuttings piles, it would be necessary to place impermeable membranes on a layer of sand sufficiently thick to cover all the debris.
2) Placement of impermeable membranes directly on top of cuttings would certainly result in the disturbance of the surface materials and loss of contaminated material to the water column. Such losses may be significant if the pile surface comprises the very soft 'gelatinous' layer which has been reported to occur on some piles.
3) Membranes have a limited lifespan of approximately 30 – 50 years (Vellinga, 1989[1]; Csiti, 1993[2]), particularly in water.

For all of these reasons, there appears to be little merit in further consideration of synthetic membranes and they are not carried forward for further assessment.

4.3.2 Permeable membranes

Permeable membranes could be used to prevent or reduce loss of solids from cuttings piles but would not prevent release of contaminated pore water. The difficulties of installation are broadly similar to those for impermeable membranes except that the requirements for joining, and for sealing at the edges and against structures, are less onerous. The merits of permeable membranes must be compared with the use of a sand layer (which would fulfil the same objective of solids retention) and appear to be very limited in view of:

---

[1] Csiti, A., 1993. *Land disposal of Contaminated Dredged Material and Related Issues: State of the Art Report.* Terra et Aqua, No. 53, December 1993, IADC, The Hague.
[2] Vellinga, T., 1989. *Land Based Disposal in the Netherlands / Case Study.* Proceedings of the International Seminar on the Environmental Aspects of Dredging Activities, Nantes, November 27 – December 1, 1989.

- the high cost of supply and placement,
- the considerable difficulties of placement especially within the footprint of existing structures, and;
- the likelihood that the surface of the cuttings will be disturbed during placement.

4.3.3 Mattresses (Protective)

Protective mattresses comprise a geogrid or geofabric on to which are fixed concrete blocks. They are typically 6.0 x 2.5 m in size and the weight can range between 150 and 500 kg/m^2. They are often used in coastal protection works as an outer layer for revetments and are used in the offshore industry to protect pipelines and for scour protection around structures. Mattresses cost in the range £20-35 (FOB) per square metre. They could be installed to protect the covered pile in situations where the pile is not obstructed by any remaining structures but would be extremely difficult, if not impossible, to place underneath structures.

The protection would be adequate for preventing damage to the cover by currents. However, they would almost certainly be damaged by anchors and, possibly, by trawling gear. This risk would be enhanced by the fact that the mattresses would need to be laid in an overlapping pattern (by 15-20%) in order to ensure full coverage, bearing in mind the obvious difficulties of placement in deep water. Snagging of the mattresses could result in significant removal of the cover protection. They compare poorly with the alternative of armour stone which is relatively cheap and easy to place.

4.3.4 Concrete

Concrete might be used in several ways:

1) placed directly over an initial layer of sand or gravel to act as the main protection layer;
2) pumped into geotextile containers to form the main protective layer;
3) placed directly over rock armour to provide additional strength and resistance to impacts (trawling, structural collapse, anchoring);
4) placed over rock armour to make the surface more ‘trawling friendly’.

Concrete placed directly on a (fine) granular substrate may damage the substrate by flow. It would be difficult to place accurately (in terms of coverage and thickness) and, while it may provide protection against current action and trawling impacts, it would not provide adequate protection against anchoring impacts unless laid in a very thick layer. There appears to be little merit in its use in this manner. Pumping concrete into geotextile containers to form a rigid mattress would be difficult, particularly in deep water, because of difficulties in controlling concrete placement into relatively small containers and because of the difficulty of initially placing the geotextile containers without disturbing the cuttings pile.

Placement of concrete on top of a primary rock armour layer to provide additional resistance may be easier but appears to have little merit. The primary armour should already be designed to provide adequate protection against currents and trawling and addition of concrete would be unlikely to increase its resistance to more severe impacts such as anchoring and structural collapse. Using concrete to smooth the external surface of the armoured pile in order to provide a ‘trawling friendly’ surface may have some merit.

However, as concrete would constitute an effectively impermeable layer, the objections raised in the previous discussion of impermeable membranes also apply here, *ie*. it would prevent dissipation of excess pore pressures and may lead to cuttings instability under additional static and dynamic loads.

4.3.5 Mattresses (Absorbent)

Research into methods of containment of contaminated dredged materials has shown that certain soils, particularly clays and peats, have the ability to fix contaminants and thus effectively act as filters which reduce the concentration of contaminants in fluids which pass through them. Use of natural materials with these qualities has been tested in onshore containment facilities (eg, de Vos et al., 1997[1]) and, in theory, could be adapted for use in pile covers. The use of mattresses containing contaminant-absorbing materials would be difficult and costly and would only have merit if it were shown that fluid contaminant loss from a sand cover is unacceptable.

4.3.6 Other materials

In the past, it has been suggested that materials such as grout, asphalt and resins could be used to fill voids in granular materials (eg. armour stone). They could in theory be used to reduce the permeability of a cover comprising granular materials, if necessary, but this again raises the risk of instability because of reduced pore water pressure dissipation. They are subject to a number of uncertainties and potential problems including:

- it would be difficult to control the hardening process (cooling in the case of asphalt) with the consequential risk of blockages of placement system (probably a fall pipe vessel or similar) particularly in the event of unexpected delays;
- hardening control difficulties could alternatively give rise to the possibility that the materials are washed away after placement;
- there are uncertainties concerning long term behaviour;
- some components may represent an environmental burden either immediately or in the long term.

4.4 SCREENING

This initial review of construction materials and methods confirms the findings of the Phase I - Task 5.1 report, ie. that existing methods of construction and considerations of effectiveness and cost require covers to be constructed using natural granular materials and that they should comprise three main layers:

1) an initial layer of sand which can be placed gently on the cuttings with minimum disturbance; the sand layer serves to accommodate contaminated pore water expelled from the cuttings and provides a suitable foundation for coarser protective materials;
2) a layer of gravel laid on top of the sand to serve as a filter layer between the sand and the outer armour and to minimise damage to the sand layer during placement of the armour;
3) an outer layer of stone armour.

It appears technically feasible to construct variants of this basic design, eg:

1) include contaminant-absorbing mattresses, perhaps as intermediate layers between the sand and gravel for cuttings piles where the sub-structure has been substantially removed;
2) cover the armour layer with concrete;
3) smooth the outer armoured surface using gravel to reduce the risk of snagging fishing gear;
4) modify the texture of the outer surface in some way for habitat enhancement purposes.

It is shown later in this report that the use of contaminant absorbents is likely to be unnecessary. Adding a concrete outer layer serves little purpose and may hinder the dissipation of excess pore water pressures within the cuttings, thus increasing the risk of instability under dynamic loads. Both of these design variants are therefore discounted. Textural modifications to the outer surface are relatively easy to achieve and are retained for further examination.

[1] de Vos, K., W. van Crombrugge, and B. Malherbe (1997). *Environmental Impact of the storage of contaminated dredged material - two case studies*. In: International Conference on Contaminated Sediments. September, 1997. Rotterdam, The Netherlands. Pre-prints. pp. 981-991.

4.5 CONSTRUCTION OF GRANULAR PILE COVERS

If the structure has been completely or largely removed, the construction of a pile cover using granular materials is relatively straightforward and can be undertaken using one or more of three main items of plant:

1) fall pipe vessels;
2) trailing suction hopper dredgers;
3) side dump vessels.

These vessels, and their general capabilities and limitations, are described below. Constructing a cover underneath a largely intact structure will be much more difficult and options for this scenario are considered separately in *Section 4.5.4*.

4.5.1 Fall pipe vessels

Fall pipe vessels (FPVs) are well known and widely used by the offshore industry to place granular materials for a variety of purposes including protection and insulation of pipelines, scour protection and platform ballasting. In essence, they comprise a dynamically-positioned ship with large holds for the transport and storage of the materials which are to be placed. The materials are placed in position using the 'fall pipe' which is usually located near the centre of the ship and is built up in sections as it is lowered to the required depth. An ROV is attached to the lower end of the pipe and is equipped with thrusters which enable the position of the fall pipe outlet to be varied within a limited horizontal range which is dependent on the length of pipe which has been deployed. However, the main method of moving the pipe during discharge is to use the vessel's DP system. Vertical positioning is achieved using the hoisting wires of the ROV in combination with a heave compensation system. The ROV is equipped with cameras to monitor operations.

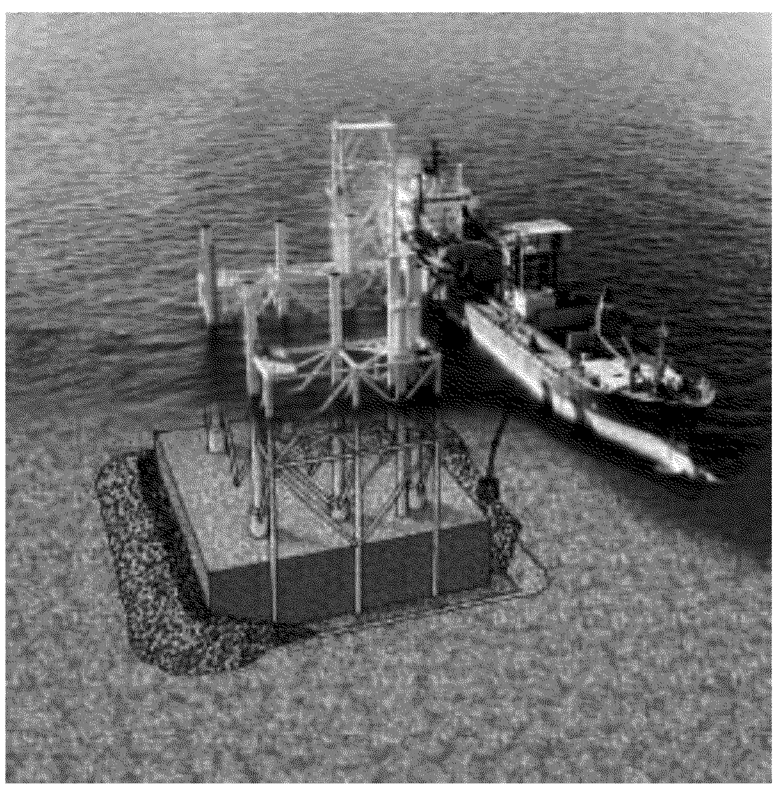


Figure 4.1. A fall pipe vessel placing scour protection

The materials are loaded into the fall pipe using a system of conveyors and cranes or excavators and are allowed to fall under gravity. The vessel moves forward during discharge, at a speed of up to about 0.2 ms^{-1}, laying materials in a series of parallel, slightly overlapping strips. FPVs are generally considered to be the most suitable method of accurate placement of granular materials at depth but are subject to certain limitations. The maximum size of stone which can be placed varies from vessel to vessel depending on a combination of fall pipe diameter and the limitations of the materials handling systems. It is generally of the order of 300 - 400 mm but at least one vessel can handle materials up to 500 mm. Sand can be placed using FPVs but, due to the relatively slow settling velocity, there exists a risk of pipe blockage unless placement is carried out slowly. This risk may be reduced (and efficiency increased) if fine gravel or a mixture of coarse sand and fine gravel is used.

The operational depth of FPVs varies but all cuttings piles in the study are within the depth capabilities of most FPVs, some of which can work at depths far in excess of 200 m. Discharge can proceed efficiently in wave heights up to about 3 metres, the limiting wave height being about 3.5 m. Currents should not exceed 1 ms^{-1} during discharge.

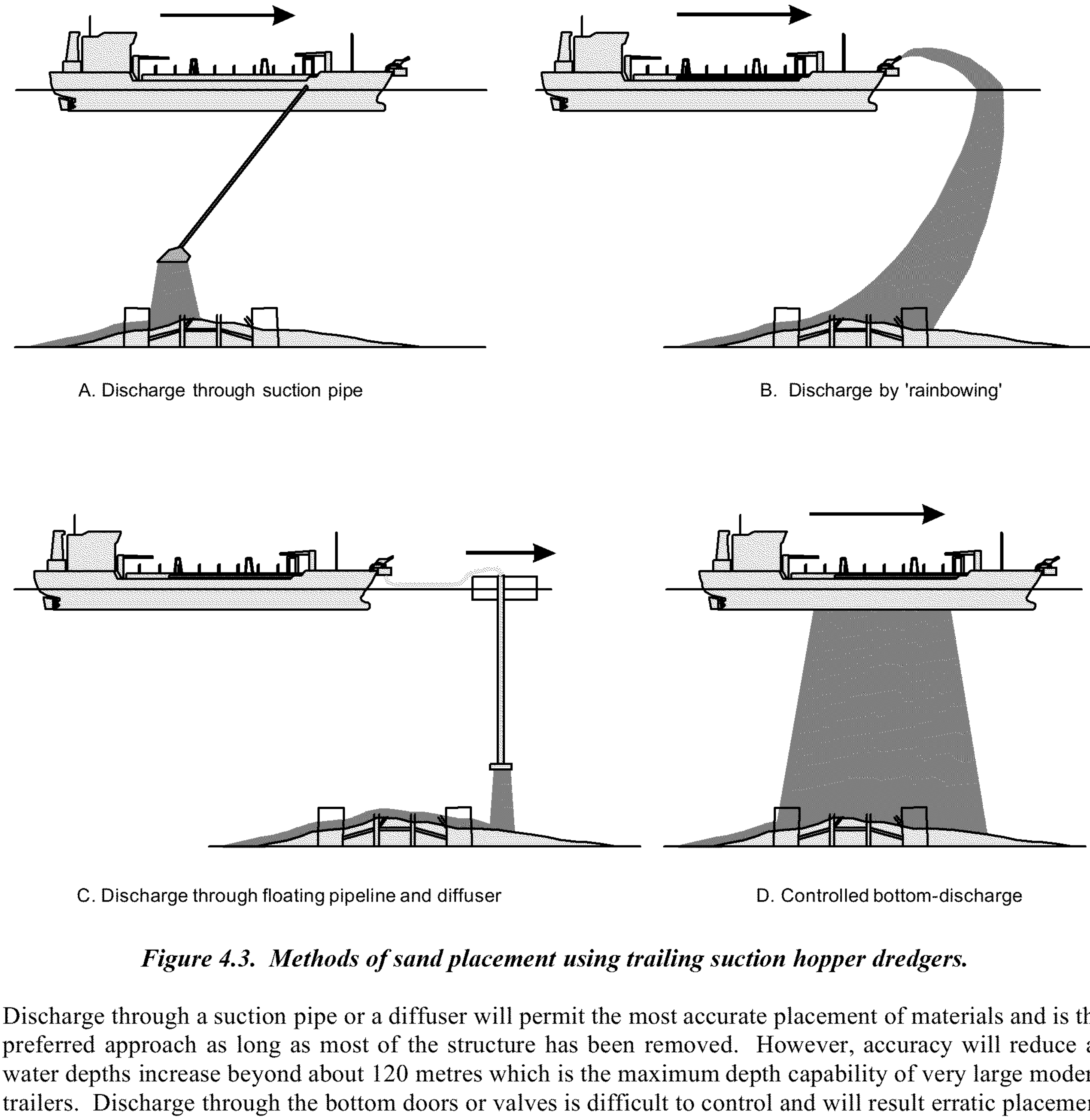
4.5.2 Trailing suction hopper dredgers

Trailing suction hopper dredgers (THSDs) are one of the most common types of dredger used by the dredging industry for work in soils and are frequently used by the offshore industry, mainly for pipeline trenching and burial. They comprise a seagoing ship with a large hopper which accommodates the dredged materials. The capacity of the hopper ranges from 100-200 m^3 up to 33,000 m^3. The materials to be dredged are raised from the bed into the hold through one or, more usually, two suction pipes. At the lower end of each pipe, a specially shaped 'draghead' moves over the seabed and maximises the entrainment of material into the suction pipe. The draghead may be fitted with water jets or blades to enhance excavation capacity. The pumps which are used to raise the mixture of soil and entrained water are located in the hull of the vessel but there are several vessels which are also equipped with pumps mounted on the suction pipe to enhance dredging capability when working at great depths. Depending on size, THSDs can work at maximum depths of between about 15 metres and 120 metres, the greatest depths being reached only by the largest, most modern vessels.

Figure 4.2. A large trailing suction hopper dredger. One of the two suction pipes can be seen lying inboard on the port side under the three deployment gantries. The equipment on the bow permits discharge by 'rainbowing' or connection to a floating pipeline.

THSDs can be used to dredge (and place) sand and very fine gravel. They would not be suitable for work with coarse gravels or armour stone. Sand and fine gravel could be placed by a THSD using one of four methods:

1) discharged deep in the water column through one of the suction pipes, the depth being dependent on the size of the vessel (*Figure 4.3A*); the suction pipe may be fitted with energy-dissipating plates and a diffuser to reduce exit velocities;
2) sprayed ('rainbowed') over the sea surface through a bow-mounted nozzle and allowed to settle onto the cuttings pile (*Figure 4.3B*);
3) pumped through a bow connection into a floating pipeline leading to a diffuser arrangement located some distance from the dredger(*Figure 4.3C*);
4) discharged through doors or valves set in the bottom of the hold and allowed to settle onto the pile. (*Figure 4.3D*).



A. Discharge through suction pipe

B. Discharge by 'rainbowing'

C. Discharge through floating pipeline and diffuser

D. Controlled bottom-discharge

Figure 4.3. Methods of sand placement using trailing suction hopper dredgers.

Discharge through a suction pipe or a diffuser will permit the most accurate placement of materials and is the preferred approach as long as most of the structure has been removed. However, accuracy will reduce as water depths increase beyond about 120 metres which is the maximum depth capability of very large modern trailers. Discharge through the bottom doors or valves is difficult to control and will result erratic placement rates and disturbance of the cuttings. Rainbowing over the bow would be effective if the ship is able to sail over the cover site but the materials may spread over a wide area during descent to the bed. Wastage may be high and it would not be possible to place material accurately in small areas. However, it may be possible to place materials under an existing structure in this manner. Discharging through a floating pipeline to a diffuser arrangement offers no technical advantages when compared with an FPV but it may be an option for placement under a structure. There may be a cost advantage if the dredger could source the sand from a nearby location but this may well be offset by the cost of the diffuser arrangement.

4.5.3 Side dump vessels

Side dump vessels (*Figure 4.4*) are used to place coarse stone and do so simply by pushing the material over the side of the vessel using hydraulic 'dozer' blades. Because placement is from the surface, the materials, especially will spread during descent, accuracy of placement would not be great and there would be some wastage of material. Placement accuracy can be improved by the use of real-time acoustic measurement of the current profile at the site which is used to adjust the position of the vessel. High accuracy placement has been achieved in this manner in water depths of up to 85 metres but, inevitably, difficulties increase with increasing depth. The advantage of side dump vessels is that they can be used to place armour which is much coarser than can be handled using fall pipe vessels.

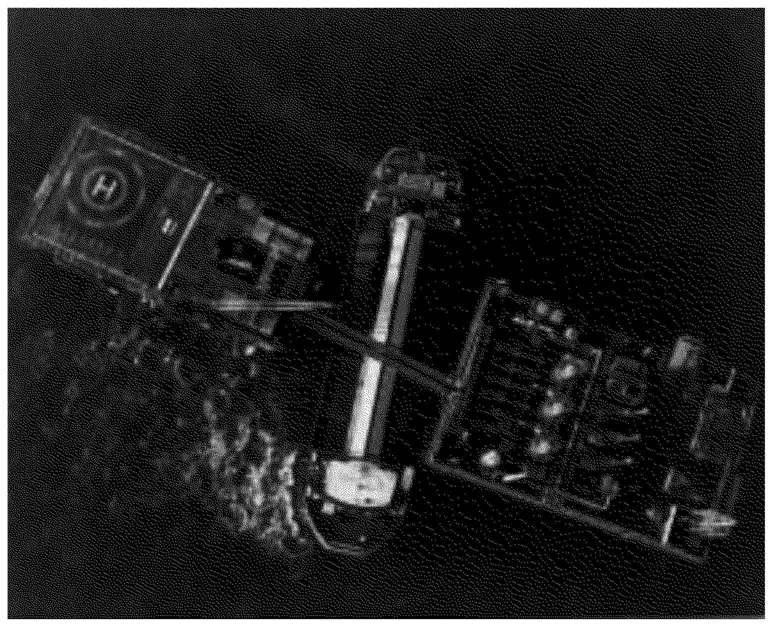


Figure 4.4. A side dump vessel underway (left) and placing scour protection between two steel installations (right).

4.5.4 Placement under existing structures

Fall pipe vessels, trailing suction hopper dredgers and side dump vessels can all be used to place covering materials with little difficulty if the structure has been removed, even if a part of it (eg. the footings and lower members) remains protruding above the cuttings pile. The difficulty of construction increases significantly if the cover is to be placed under an existing structure. The main options are briefly reviewed here for each material type.

Sand

The sand layer (or very fine gravel) could be placed using trailing suction hopper dredgers by rainbowing the material under the topsides. This method can be used to project sand up to 60 metres ahead of the vessel's bow. Accuracy of placement on the pile would be extremely difficult to control (*Section 4.5.5*). It would also be possible to pump the sand as a slurry to a small diffuser pontoon located under the topsides which is winched slowly across the area to be covered. The practicality of this option will depend largely on the design of individual structures and difficulties of access. A third approach is to use chutes attached to the structure but, although this has been suggested by previous studies, this approach is considered to be fraught with problems due mainly to the difficulty of moving the chute during discharge and interfacing with the vessel supplying the sand. It is likely to be prohibitively expensive.

Fall pipe vessels and THSDs could be used to discharge sand adjacent to the structure, relying on currents to move the material under the structure as it descends to the pile. This practicality of this technique would depend on the current regime at the site in question and not all sites would be suitable. Control and accuracy of placement would be extremely low.

A completely different approach, suggested during Phase I of the JIP, is to place the material in a two-stage operation involving the use of ROVs (*Figure 4.5*). The sand would be placed in one or more temporary stockpiles on the seabed using FPVs. It would be excavated from the stockpile and pumped to a workclass ROV which would discharge it at the required location on the pile. The benefits of this approach lie in the high degree of placement control which can be exercised but it would clearly be a costly method and production rates would be very low.

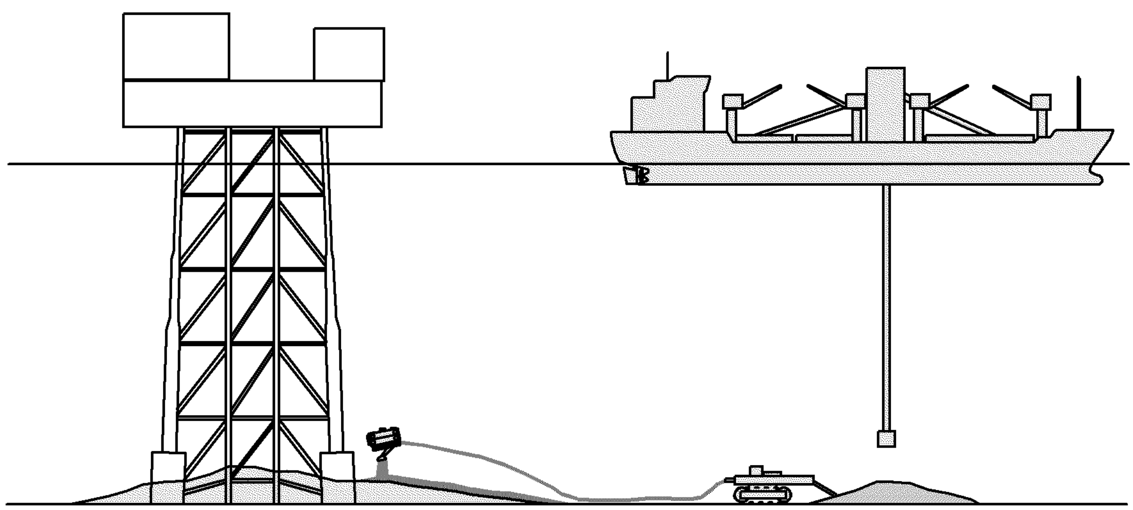


Figure 4.5. Placement of sand under a structure using a fall-pipe vessel and ROV.

Gravel

Because of its very high fall velocity in water, it would be extremely difficult, if not impossible, to place gravel under most structures by discharging at or near the surface and relying on currents to move the gravel under the structure. Using THSDs to pump to diffuser pontoons or to rainbow under the structure is also not a practical solution because the systems required to discharge the hopper in this manner are not capable of working efficiently with materials larger than very fine gravel. A two-stage placement approach using seabed stockpiles and ROVs may be technically possible but the size of the gravel is likely to be a problem for the stockpile excavation and transport systems and production rates would be wholly unacceptable. On balance, it is considered that placing the gravel layer under an existing structure is impractical.

Armour stone

Placing armour stone under an existing structure is not considered to be practical and is discounted.

4.5.5 Accuracy of material placement

The accuracy with which materials can be placed using the vessels described above is important in the context of the design of covers and the estimation of cost of construction. Placement accuracy is important with respect to:

1) the minimum thickness of material which can be placed in a single ‘pass’; this may be relevant to stability when covering, potentially unstable cuttings and it defines the minimum nominal thickness of each component of the cover;
2) the additional thickness of material which must be placed (over and above the design thickness) in order to ensure that the required thickness is actually achieved (this is mainly relevant to construction cost and duration);
3) the horizontal accuracy which material can be placed, eg. is it possible to vary accurately the thickness of material over different parts of the pile in order to smooth the contours of an irregular cuttings piles and to achieve a particular finished profile?

Fall pipe vessels

When working directly over the areas where materials are required, fall pipe vessels can place material with a high degree of horizontal accuracy and could be used to modify the contours of the piles during placement. The minimum thickness which can be placed in a single pass is theoretically about 0.3 m but several factors need to be considered when determining what is actually achievable in the field:

- there is a time lapse between material entering the fall pipe and discharging at the bottom which means any adjustments to placement rates which are required to accommodate variable pile topography are subject to delay;
- there is time lapse between the ROV being pushed aside by the currents and restoring its position;
- material is discharged from a height of about 4m above the seabed and there is no control over the movement of individual particles during their descent between the fall-pipe outlet and the seabed;
- although the final overall shape of the placed material will be in good agreement with what may be expected in terms of height and side slopes on the basis of the material properties, adjustment of the slope of piles, if required to modify their shape (eg. for stability reasons) will require extra passes;
- it is necessary to make an allowance for overlapping of adjacent strips of placed material.

Taking all these factors into consideration, it seems realistic to assume that, in practice, the minimum nominal design thickness of each of the layers of sand, gravel and armour should be taken as 1 metre.

If the structure is in place during covering and the FPV must rely on currents to move material under the structure, the accuracy of placement will inevitably be very low. In the case of armour stone and gravel, this approach is deemed to be impractical. For sand placement, it may work in areas where there is sufficient current but it would certainly not be possible to accurately re-contour a pile of marginal stability with this method. It would be necessary to place a significant additional volume of sand in order to ensure that the minimum required thickness is achieved at all locations and it is therefore likely that some areas may receive far more material than is required. In some cases, this may induce pile instability.

Trailing suction hopper dredgers

Where the structure has been removed prior to covering, placement of sand (and possibly fine gravel) using THSDs will be less accurate than using FPVs. Bottom-discharge through doors or valves has been discounted because of control difficulties but even with suction pipe discharge, rainbowing or the use of floating pipelines and diffusers, it is not realistic to expect that a THSD could accurately re-contour a pile. However, they should be capable of placing relatively thin (but variable) layers over wide areas of the pile although some material waste will be inevitable particularly in the case of material released at high altitudes above the pile. With the structure in place, both horizontal and vertical accuracy of placement will suffer and wastage rates will increase. It is considered that THSDs should only be used to place cover materials in cases where accuracy is not a major consideration

Side dump vessels

These would only be used to place armour stone and then, probably, only in the event that the stone is too large to be placed using FPVs or if FPVs are not available at the time. In shallow water, a side dump vessel can place armour layers with a vertical accuracy of about plus or minus the nominal diameter of the armour. In the deep water characteristic of cuttings pile locations, accuracy will be reduced partly because of the effects of currents and partly because of inaccuracies where adjacent passes overlap. The size of armour proposed to protect against extreme environmental conditions (*Section 6*) is up to about 300 mm. At least two layers will be required and, allowing for a combination of placement inaccuracy and survey inaccuracy, a minimum nominal thickness of 1 metre appears reasonable.

4.6 CONCLUSIONS

The review of potential construction materials and methods leads to the following main conclusions:

1) pile covers should be constructed using natural granular materials;

2) the covers should comprise an initial layer of sand to accommodate pore water expelled from the cuttings and which can be placed with minimum disturbance of the cuttings; a layer of gravel laid on top of the sand provides a suitable substrate for placing the relatively large armour stone required for protection of the pile and to act as a filter layer between the sand and the armour stone;

3) proven methods of placement exist for such materials and, if most of the structure has been removed prior to covering, covers can be constructed using combinations of fall pipe vessels, trailing suction hopper dredgers and side dump vessels depending of the details of the materials to be placed and site conditions; fall pipe vessels are the preferred method;

4) while it is considered possible to place the sand under an existing structure (albeit with considerable difficulty and loss of accuracy), it is not considered practical using existing technology to place gravel and armour stone under structures;

5) consideration of typical site conditions, the nature of the available construction equipment and limitations of survey methods leads to the conclusion that the practical minimum thickness of the sand, gravel and armour stone layers which can be placed should be taken to be 1 metre; detailed design considerations may indicate a requirement for greater thicknesses but this does not constitute a constraint.

5 WAVES AND CURRENTS

5.1 INTRODUCTION

The wave and current climates have implications for both the design and construction of covers. The current regime (the combination of tidal and wave-induced currents) defines certain minimum requirements with respect to the nature of the external armouring placed on the cover. Waves and currents also have the potential to cause pore water movement within the pile and the cover which may lead to release of contaminated leachate. They may also have a bearing on the method of construction and on delays and overall cost of construction. This section reviews the wave and current regimes in the UK Northern and Central Sectors, leading to the derivation of the following scenarios relevant to cover design:

- average conditions which are used in the detailed review of construction costs (*Section 11*).
- extreme conditions which are used to identify the minimum requirements for armouring.

Three locations have been selected for analysis:

- Northern Sector at approximately 1° 20'E, 61° 40'N, where the depth of water is about 186m;
- Central Sector at approximately 1° 10'E, 57° 50'N, where the depth of water is about 120m;
- the Moray Firth at approximately 3° 6'W, 58° 12'N, lying within the Central Sector but relatively sheltered and with a shallow water depth of about 46m.

For the locations in the Northern and Central Sectors, measured wave, current and surge data were made available to the Study Team by Shell U.K. Limited[1]. For the Moray Firth location (Beatrice), data were synthesised from information provided in Department of Energy, (1990)[2].

Typical 'shallow' and 'deep' water conditions are assessed in both the Northern and Central Sectors. In the Northern Sector, the water depth varies between about 100m and 200m, and the distribution of the piles in this Sector, in relation to the depth of water, is shown in *Figure 2.5*. 'Deep' water is taken as 170m, representing piles in 150m to 190m of water; and 'shallow' water is taken as 120m, representing piles in 100m to 150m of water. In the Central Sector, the water depth varies between about 40m and 160m. The distribution of piles in this Sector is also shown in *Figure 2.5*. 'Deep' water is taken as 140m, representing piles in 110m to 160m of water; while 'shallow' water is taken as 90m, representing piles in 70m to 110m of water. In the Moray Firth, the three Beatrice installations are all in about 46m of water, as noted above.

5.2 WAVES

5.2.1 Available data

The data made available by Shell included the following:

- monthly percentage exceedence for significant wave height for the period 1 January 1975 to 31 December 1994;
- seasonal persistence for significant wave height for February-March, April, May-August, September-October, and November-January;

[1] The terms under which the information was provided to the Study Team include a requirement to reproduce the following disclaimer: *"The information contained herein is provided to* (Dredging Research Limited) *by Shell U.K. Limited and Shell U.K. Limited makes no representation or warranty (whether express or implied) about and shall have no liability whatsoever (other than in the event of fraud) for the accuracy or completeness of any information contained herein. Any use of or reliance upon any information contained herein shall be at the user's own risk. Shell U.K. Limited's liability for fraud is not excluded."*
[2] Department of Energy, 1990. *Offshore Installations: Guidance on Design, Construction and Certification. Section 11 – Environmental Considerations.* HMSO, London, 1990

- results of extreme value analyses of significant wave height (3-hr), maximum wave height (3-hr), and wave period (T_z, T_p and T_{ass}, with lower, central and upper values in each case) for the 1/12-year, 1-year, 5-year, 10-year, 50-year and 100-year return periods.

The monthly percentage exceedence for significant wave height confirms that the highest waves occur in winter, and particularly:

- in the Northern sector during the period October to January, when significant wave heights of over 11.0m can occur, with associated periods in the range 12s to 15s; and
- in the Central sector during the period October to December, when significant wave heights of over 10.0m can occur, with associated periods in the range 11s to 14s.

50-year return period significant wave heights are:

- 15.7m in the Northern sector with associated periods in the range 14s to 18s,
- 12.8m in the Central sector with associated periods in the range 13s to 16s.

For the Moray Firth location, Department of Energy (*op cit*) indicates a 50-year return period significant wave height of about 10.5m. The same source indicates a 50-year return period significant wave height of about 14m in the Central sector which is higher than that indicated by the Shell data. However, it is understood that Department of Energy (*op cit*) gives significant wave heights that are considered conservative.

5.2.2 Design waves for pile covering

It is considered that for the design of the covering to the piles, a 500-year return period significant wave height should be taken as representing the design wave conditions. To obtain this for the Northern and Central sectors, an extreme value analysis was carried out, using the annual percentage exceedence figures for the two locations. The results of this were compared with the return period figures provided by Shell.

In the case of the Central sector, the figures compared well, and the 14.4 metres indicated by the analysis for the 500-year return period significant wave height was adopted. In the case of the Northern sector, the analysis suggested slightly lower return period significant wave heights than those provided by Shell. The figures from the analysis were therefore adjusted upwards by appropriate factors to provide a better comparison with the Shell figures, and an adjusted 500-year return period figure of 17.7 metres was adopted.

For the Moray Firth, no monthly or annual percentage exceedence figures, or return period significant wave heights were available. Since the Central sector and Moray Firth locations are at approximately the same latitude, it was considered reasonable to scale down the exceedence figures for the Central sector in the ratio of the 50-year return period significant wave heights indicated for Beatrice and for Central North Sea in Figure 11.3 of Department of Energy (*op cit*) and then to apply an extreme value analysis to the figures reduced in this ratio. From this analysis, a 500-year return period significant wave height of 10.8 metres was adopted for the Moray Firth.

To obtain the wave period associated with the 500-year return period significant wave heights (T_{ass}) for the Northern and Central sectors, the ratio of T_{ass}(central)/(H_s)^0.5 for the 10-year, 50-year and 100-year return periods data were extrapolated to give the 500-year return period figure for T_{ass}(central).

It is important to consider the probable range of wave period associated with the 500-year return period significant wave height, since many effects due to waves are sensitive to this parameter. The figures for T_{ass}(upper) and T_{ass} (lower) were obtained by multiplying T_{ass} (central) by the constant ratio T_{ass} (lower)/ T_{ass} (central) and T_{ass} (upper)/ T_{ass} (central) obtained from the Shell return period T_{ass} data. For the Moray Firth, the figures for T_{ass}(central), T_{ass} (upper) and T_{ass} (lower) were obtained from the 500-year significant wave height in a similar manner, by assuming the same ratios used for the Central sector for the 500-year return period figures.

The 500-year return period wave parameters which have been adopted are summarised in *Table 5.1* below. It should be noted that these are 'all directions' parameters.

Table 5.1 Summary of Design 500-year return period wave parameters

Wave Parameter	*Location*		
	Moray Firth	*Central Sector*	*Northern Sector*
Significant wave height, H_s	10.8m	14.4m	17.7m
Wave period, T_{ass} (lower)	12.0s	14.0s	15.0s
Wave period, T_{ass} (central)	13.5s	15.7s	16.8s
Wave period, T_{ass} (upper)	14.8s	17.3s	18.6s

5.2.3 Differential subsurface pressures at the pile due to passing waves

For consideration of differential subsurface pressures at the pile as the wave passes, leading to possible 'pumping' action, it is primarily the annual wave climate that is of interest, and the percentage of time that waves of various significant height can be expected to prevail. These data can be obtained directly from the monthly and annual exceedence figures. In this case, the wave period associated with each bandwidth of wave height was taken to be given by T_z*1.2, where $T_z = 11*(H_s/g)$^0.5, where H_s is the average significant wave height in the bandwidth.

The differential subsurface pressures at the pile as the wave passes under the 500-year return period conditions has also been assessed. The maximum wave-induced pore water pressure increases in the cuttings pile are relevant to their overall stability.

5.2.4 Construction constraints

The seasonal persistence figures for significant wave height made available by Shell for the Northern and Central sectors provide the data which are required for the assessment of construction constraints and delays. For the Moray Firth, an approximate assessment can be made by scaling down the Central sector significant wave heights in the ratio of the 50-year return period significant wave heights indicated for Moray Firth and Central sector in Figure 11.3 of Department of Energy (*op cit*) as was done for the 500-year return period wave heights in the Moray Firth, described above.

5.3 WATER LEVELS

5.3.1 Available data

The Shell data for the Central and Northern sectors included the following:

- tidal heights relative to Lowest Astronomical Tide (LAT) level; these heights do not vary with return period;
- results of extreme value analyses of positive and negative surge for the 1/12-year, 1-year, 5-year, 10-year, 50-year and 100-year return periods.

5.3.2 Extreme positive surge

The 500-year return period positive surge has been estimated by considering the ratio 50-year surge/5-year surge, applying this factor to the 50-year surge, and comparing this with the ratio 50-year surge/10-year surge applied to the 100-year surge. These give similar results. By this method, the 500-year return period positive surge for the Northern sector approximates to 0.86 m, and for the Central sector to 1.80 m.

From Figure 11.5 of Department of Energy (*op cit*) the 50-year return period positive surge at the Moray Firth location is about 1.25m. Applying the same factor to this 50-year surge as for Central sector to obtain the 500-year surge at Beatrice yields a 500-year return period positive surge of 1.53m.

5.4 TIDAL AND STORM SURGE CURRENTS

5.4.1 Available data

The Shell data for the Central and Northern sectors included the1/12-year (Central sector only), 1-year, 5-year, 10-year, 50-year and 100-year return period total current profiles, expressed in m/s at various levels below the surface, together with the depth-averaged total current. It was assumed that the 'total current' is that due to tidal current plus the component due to storm surge.

5.4.2 Design total current

Currents due to tide and the 500-year return period positive surge were estimated by considering the ratio [50-year total current]/[5-year total current] at the required level in the profile, applying this factor to the 50-year total current, and comparing this with the ratio [50-year total current]/[10-year total current] applied to the 100-year total current. These give similar results.

A typical level of the surface of the pile is taken to be 5m above the sea bed. Thus the profile level of 0.05d, where d is the depth of water, approximates to this level, except at Beatrice where 0.10d approximates to the surface of pile level. The estimated 500-year return period total current velocities at this notional top of pile level at the three locations are as follows:

Moray Firth (Beatrice):	0.49 m/s
Central Sector:	0.78 m/s
Northern Sector:	0.76 m/s

6 COVER PROTECTION REQUIREMENTS

6.1 INTRODUCTION

The pile cover must maintain its integrity under a number of loading scenarios. These are:

- extreme environmental loading due to waves and currents;
- disturbance by ships' anchors and fishing gear;
- impacts of pieces of the installation breaking off, over time, and ultimately, larger portions of the installation collapsing onto the pile.

The requirements for protection of the covering to piles are considered in the light of each of these types of loading. In addition, this section reviews the implications for disturbance (both deliberate and accidental) of the cuttings during structure decommissioning.

6.2 PROTECTION AGAINST EXTREME ENVIRONMENTAL LOADING

6.1 Long term protection

There several methods which can be used to assess the size of underwater rock armouring required to withstand the loading from the orbital motion of waves and from currents near the sea bed. These include methods due to Isbash (1970)[1], Rance and Warren (1968)[2], Pilarczyk (1990)[3], Soulsby (1997)[4], Maynord (1993)[5], and Escarameia and May (1992)[6]. Of these, the method due to Isbash is probably the most appropriate and is suggested by BS 6349: Part 7: 1991[7].

The slopes of the pile surfaces have been reviewed in *Section 2.3*. These range from very shallow slopes near the edges of the piles, typically 1° to 5° (1:57 to 1:11), to much steeper slopes nearer the centre of the pile, typically 26° to 37° (1:2 to 1:1.3). In practice, although the steeper slopes approximate to the range of the natural angle of repose of granular covering material, the size of rock armouring required to remain stable under the 500-year return period conditions, and the difficulty of accurately placing rock armour on such slopes at depths of up to 200 metres, suggest that it may be better to form flatter slopes. The rock size on slopes greater than 18° (1:3) would at many locations be at or beyond the handling capability of most fall pipe vessels and 18° has therefore been selected as the indicative maximum design slope. Re-contouring of cuttings piles which exhibit slopes steeper than 18° slopes could be achieved by adjusting the thickness of the initial sand layer before placing the armour layers.

Considering the 500-year return period conditions, and taking into account the possible range of periods associated with the 500-year significant wave height, it is found that the higher the period, the greater must be the mass of the armouring. *Table 6.1* lists the nominal rock armour masses and sizes (considering the rock as a sphere) which are required to ensure stability. These have been computed for wave periods at the upper end of the probable range.

[1] Isbash, S V & Khaldre, K Y., 1970. *Hydraulics of River Channel Closure*. Butterworths, London.
[2] Rance, P J & Warren, N F., 1968. *The Threshold of Movement of Coarse Materials in Oscillatory Flow*. Proc. 11th Conference on Coastal Engineering, American Society of Civil Engineers.
[3] Pilarczyk, K W., 1990. *Stability Criteria for Revetments*. Proc. National Conference on Hydraulic Engineering, American Society of Civil Engineers.
[4] Soulsby, R.L., 1997. *Dynamics of Marine Sands: A Manual for Practical Applications*. Report SR 466, HR Wallingford, Wallingford.
[5] Maynord, S.T., 1993. *Corps Riprap Design Guidance for Channel Protection.* Paper 3 of River, Coastal and Shoreline Protection: Erosion Control using Riprap and Armourstone, International Riprap Workshop, Fort Collins, July 1993, John Wiley, Chichester.
[6] Escarameia, M, and May, R.W.P., 1992. *Channel Protection: Turbulence Downstream of Structures*. Report SR 313, HR Wallingford, Wallingford.
[7] British Standards Institution, 1991. *British Standard Code of Practice for Maritime Structures BS 6349: Part 7: 1991 - Guide to the design and construction of breakwaters*. HMSO, London.

Table 6.1 Rock armour requirements for protection against extreme environmental conditions

Finished slope	Nominal weight in kg and (nominal size in mm) of armour — Location and Depth of water				
	Beatrice 46m	Central North Sea 90m	Central North Sea 140m	Northern North Sea 120m	Northern North Sea 170m
Slope 1^o (1:57)	8.1 (185)	6.8 (175)	0.7 (80)	5.2 (160)	0.7 (80)
Slope 5^o (1:11)	10.9 (205)	9.1 (195)	0.9 (90)	7.0 (180)	1.0 (90)
Slope 7^o (1:8)	12.6 (215)	10.5 (205)	1.1 (95)	8.1 (185)	1.1 (95)
Slope 11^o (1:5)	18.2 (245)	15.2 (230)	1.6 (105)	11.8 (210)	1.6 (105)
Slope 18^o (1:3)	40.6 (320)	33.9 (300)	3.5 (135)	26.2 (275)	3.6 (140)
Slope 26^o (1:2)	160.9 (505)	134.3 (475)	13.8 (215)	103.8 (440)	14.3 (220)

In practice, the armour would comprise a range of sizes around the nominal size. The indicated sizes may be somewhat conservative but are useful in establishing potential limitations of construction methods. The actual size of armour placed on an individual cuttings pile will depend on a detailed analysis of the pile morphology and stability and the hydrodynamic conditions at the location of the pile.

6.2.2 Protection during construction

The near-bed wave peak particle velocities at each location were calculated as part of the assessment of the size of the required covering materials. The peak particle velocities under the 500-year return period conditions were assessed as set out in *Table 6.2*.

Table 6.2 Estimated wave peak particle velocities, 500-year event.

	Peak particle velocity (m/s)				
	Beatrice 46m	Central North Sea 90m	Central North Sea 140m	Northern North Sea 120m	Northern North Sea 170m
With T_{ass} Upper	1.84	1.49	0.77	1.41	0.80
With T_{ass} Central	1.68	1.27	0.57	1.15	0.57
With T_{ass} Lower	1.42	0.98	0.36	0.84	0.35

Under the 500-year return period conditions, the armour rock as designed will be stable, but not the sand and gravel layers which may be exposed to storm events during construction of the cover. Although the risk of a 500-year event occurring during the relatively short cover construction period is small, even a 1-year event could result in damage of a part-completed cover. The peak particle velocities under these conditions are assessed as set out in *Table 6.3*.

Table 6.3 Estimated wave peak particle velocities, 1-year event.

Location and water depth	Peak particle velocity (m/s)				
	Beatrice 46m	Central North Sea 90m	Central North Sea 140m	Northern North Sea 120m	Northern North Sea 170m
With T_{ass} Upper	0.98	0.62	0.22	0.59	0.25
With T_{ass} Central	0.83	0.46	0.13	0.41	0.14
With T_{ass} Lower	0.63	0.29	0.06	0.24	0.06

The size of the cover materials required to resist damage due to a 1-year return period event are set out in *Table 6.4*. The nominal sizes have been rounded to the nearest 5mm.

Table 6.4 Armour requirements for protection during construction (assumed 1-year event)

Slope	*Nominal weight in kg and (nominal size in mm) of armour*				
	Location and water depth				
	Beatrice	*Central North Sea*		*Northern North Sea*	
	46m	*90m*	*140m*	*120m*	*170m*
Slope 1° (1:57)	0.3 (65)	0.17 (50)	0.02 (20)	0.05 (35)	0.003 (15)
Slope 5° (1:11)	0.4 (70)	0.23 (55)	0.02 (25)	0.06 (35)	0.005 (15)
Slope 7° (1:8)	0.5 (75)	0.27 (60)	0.02 (25)	0.07 (40)	0.005 (15)
Slope 11° (1:5)	0.8 (85)	0.38 (70)	0.03 (30)	0.10 (45)	0.01 (20)
Slope 18° (1:3)	1.7 (110)	0.85 (90)	0.08 (40)	0.23 (55)	0.02 (25)
Slope 26° (1:2)	6.6 (175)	3.4 (140)	0.31 (60)	0.90 (90)	0.07 (40)

From the above, it is clear that the gravel of the cover layers would be stable only in the deeper locations and where the slopes are relatively shallow. Sand would not be stable under these conditions. The placement of the sand and gravel components of the covers should therefore take place as rapidly as possible (having regard to possible geotechnical constraints) and during a period in which conditions are forecast to be relatively benign. The sand and gravel layers should then be armoured without delay.

6.3 DISTURBANCE DUE TO FISHING GEAR

All data in this section has been taken from the UKOOA Trenching Guidelines JIP (Trevor Jee Associates, 1996[1] & 1998[2]) undertaken for a group of sponsors and supplied to the Study Team by one of those sponsors supporting Phase II of Drill Cuttings JIP.

6.3.1 Methods of trawling

The methods of fishing practiced in the North Sea which will have the greatest impact on unguarded drill cuttings piles are trawling, specifically beam trawling and otter board trawling. *Figures 6.1*, *6.2* and *6.3* illustrate these methods together with a more recent development, the twin otter board method.

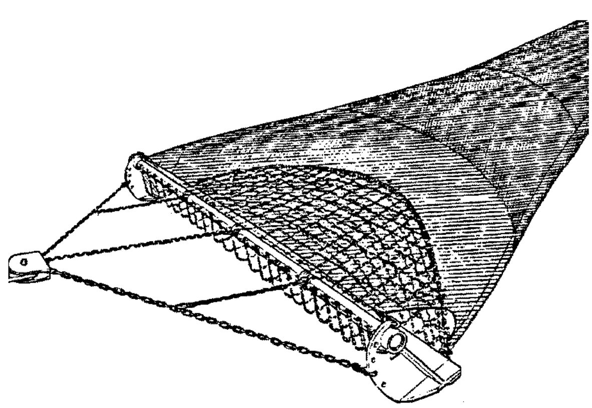


Figure 6.1 – Beam Trawl (from Trevor Jee Associates)

[1] Trevor Jee Associates, 1996. *Fishing Equipment*. Report number SH01R10A. for UKOOA Trenching Guidelines JIP, July 1996.
[2] Trevor Jee Associates, 1998. *Guidelines for Trenching*. Report number SH01R17A., for UKOOA Trenching Guidelines JIP, September 1998.

As can be seen from *Figure 6.1*, in beam trawling a rigid beam is supported above the sea bed by steel 'shoes' at each end. Chains suspended from the beam are pulled over the seabed in order to disturb the bed and lift fish into the mouth of the net. In the case of otter board trawling (*Figure 6.2*), boards or 'doors' are attached to the boat's trawl lines, with the nets attached to the doors. The position of the lines on the doors cause them to open the mouth of the net when towed. Twin rig otter board trawling is similar but with two nets and a central clump weight replacing the central doors (*Figure 6.3*).

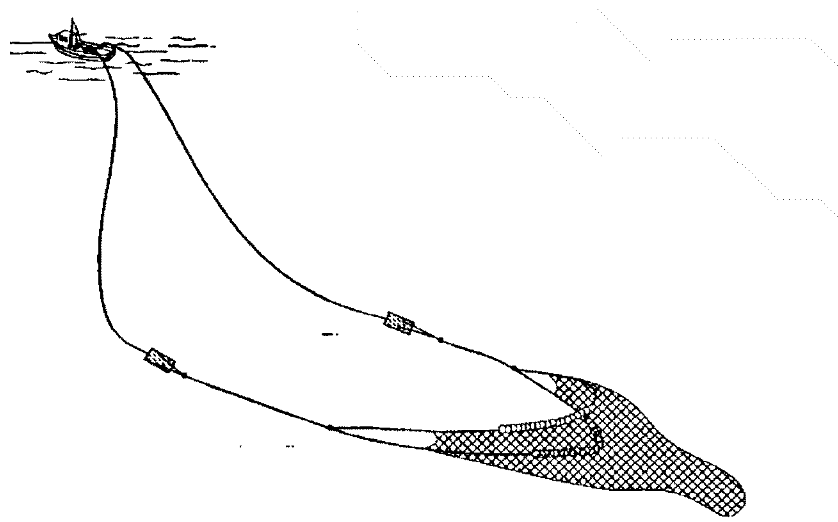


Figure 6.2 – Otter board trawling (from Trevor Jee Associates)

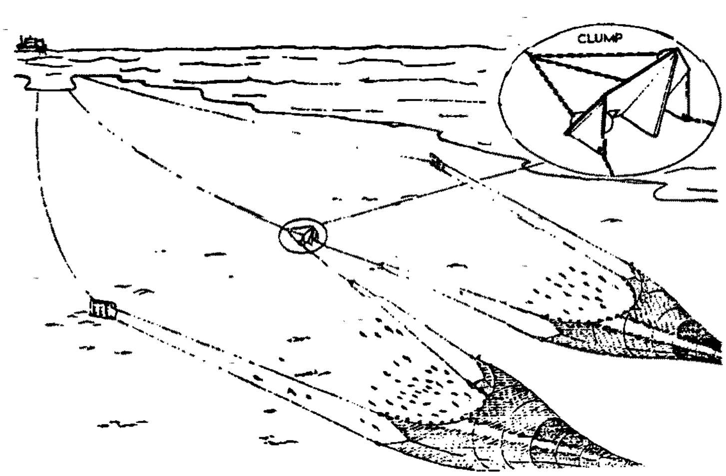


Figure 6.3 – Twin rig otter board trawling (from Trevor Jee Associates)

6.3.2 Data on trawling parameters

Beam trawls

Trevor Jee (*op cit*) provide data for Dutch beam trawl vessels and other beam trawl vessels in the North Sea (*Table 6.5*). While the Dutch vessels are bigger and have larger gear they tend to work south of 56º latitude and rarely work north of 58º.

Table 6.5 Beam trawler characteristics

Parameter	*Units*	*Dutch Vessels (Generally below 56°)*		*Other vessels (throughout North Sea)*	
		Mean size	*Largest*	*Mean Size*	*Largest*
Vessel Mass	tonnes	690	1,000	430	700
Beam Mass	kg	4,300	5,800	3,089	4,740
Mass of nets and chains	kg	2,100	2,500	1,907	2,330
Normal trawl load	tonnes force	12.0	16.0	12.0	16.0
Maximum Pull	tonnes	33	67	18	42
Trawl velocity	m/s	3.1	4.1	3.1	3.6

Otter trawls

While there are data on the physical dimensions of otter boards of the three main types (Vee, Polyvalent and Bison) there are almost no data on the trawl loads. The values quoted by Trevor Jee (*Op cit.*) are extrapolated from a single series of tests on limited data. The data used for the consideration of trawling loads on pipelines are summarised in *Table 6.6*.

Table 6.6 Otter trawler characteristics

Parameter	*Units*	*Vee doors*		*Polyvalent doors*		*Bison doors*	
		Mean size	*Largest*	*Mean size*	*Largest*	*Mean size*	*Largest*
Vessel Mass	tonnes	220	350	350	4800	220	350
Door mass	kg	640	1800	1400	2500	685	1120
Mass of net etc	kg	2500	3500	3000	4000	2500	350
Attack angle		35°	35°	15°	15°	15°	15°
Normal trawl load	tonnes	3.0	8.0	3.0	8.0	3.0	8.0
Maximum load	tonnes	5.7	30	12	57.6	5.7	12.3
Velocity	m/s	2.1	2.1	2.1	2.1	1.8	1.8

6.3.3 Design considerations for armour

The data presented in the previous section have been used to derive the indicative forces set out below by considering the distribution of mass between the beam / boards and the remainder of the fishing gear.

- *Beam trawl – below 58° latitude*
  - Vertical force at end of beam – 29 kN over length of 1.3 m
  - Horizontal (friction) force at end of beam – 15 kN over length of 1.3 m
  - Horizontal (snagging) force – 500 kN over height of 0.41 m
  - Weight of chains – 20 kN over area of say 15 m^2
  - Friction from chains – 10 kN over area of say 15 m^2

- *Beam trawl – above 58° latitude*
  - Vertical force at end of beam – 24 kN over length of 1.2 m
  - Horizontal (friction) force at end of beam – 12 kN over length of 1.2 m
  - Horizontal (snagging) force – 300 kN over height of 0.39 m
  - Weight of chains – 18 kN over area of say 15 m^2
  - Friction from chains – 9 kN over area of say 15 m^2

- *Otter trawls*
  - Vertical force from otter board – 25 kN over length of 4.6 m
  - Horizontal (friction) force from otter board – 13 kN over length of 4.6 m

The vertical forces (ie. the weight of the gear) can be accommodated by the proposed armoured cover but frictional forces will inevitably displace the relatively light armour stones which are proposed in *Section 6.2*. The magnitude of the effects are impossible to quantify numerically but a few trawling events would be most unlikely to severely damage the cover. The potential snagging forces are considerable but will not be fully realised unless the trawling gear:

- snags on extremely heavy armour; or
- drops into a hole from which it cannot be extracted.

Such events could endanger the fishing boat and its crew. The external surface of the armour should therefore be designed in such a manner as to ensure that snagging cannot occur and the gear rides smoothly over the armour pile. This leads to two design criteria:

1) the armoured pile must have smooth, regular shape without severe changes of slope or enclosed depressions;
2) the size of stone used on the surface of the armouring must be such that beam shoes and otter boards can ride over the stone.

The radii of beam shoes may vary between 0.25m and 0.35m, while the leading edges of otter boards have radii varying between 0.25 m and 1.6 m. These should be sufficient to ensure that the solid parts of the gear can ride over all of the armour stone required for cover slopes of up to 18^{o} (refer to *Table 6.1*). CUR/RWS (1995)[1] suggests that a sloping rock cover of 0.5m thickness will deflect the trawl board and will provide sufficient protection against damage (in the context of the reference, to a pipeline protected by the rock armour). There remains a risk that the nets may snag on individual stones and pluck them from the cover and consideration might be given to 'finishing' the armour layer with a sprinkling of <80mm material as is the current practice in the Dutch sector.

Despite these conclusions, it is recognised that trawling over a covered pile will result in slight displacement of some of the armour stones even if entanglement of gear and outright removal of stones does not occur. In the short term (say, a few years), this is unlikely to result in loss of cover functionality and, indeed, some damage may 'self heal'. In the medium and long term, however, the cumulative impacts of repeated trawling events could conceivably reach the point at which parts of the cover protection are removed.

This is relevant to future monitoring and maintenance requirements and, unless the cuttings degrade to a 'safe' condition, implies a long-term liability on the part of the operators. In this context, the Study Team are of the opinion that it is not realistic to expect that fishing activities in the area of covered piles can be controlled to the extent that accidental (or even deliberate) over-trawling can be guaranteed, particularly in the very long term.

It should be noted that these design criteria should be sufficient to reduce the risk of damage to trawling vessels only in the case of installations which are decommissioned to the extent that there are no structural members or large items of debris protruding above the cover. In the case of partial decommissioning, where large members may extend above the cover, it will not be possible to eliminate entirely the risk of potentially dangerous entanglement of fishing gear.

6.4 DISTURBANCE DUE TO SHIPS' ANCHORS

Very few vessels attempt to anchor in the deep waters of the North sea where the bulk of the cuttings piles occur. Those that do are primarily involved with the oil industry and can be located so that their anchors do not interfere with the piles. For such vessels, the anchor layout design takes account of seabed obstructions. It is therefore reasonable to assume that the only interaction between anchors and drill cuttings armouring will occur if such a vessel is losing location because of anchor slippage. As such vessels typically deploy

[1] CUR/RWS, 1995. Report 169: *Manual on the Use of Rock in Hydraulic Engineering*. A A Balkema, Rotterdam.

between 8 and 12 anchors at any time, and frequently have tugs and/or thrusters to assist in manoeuvring, it is unusual for an anchor to slip away from its intended route. Anchoring impacts from other vessels are thus only likely to occur in the event of emergencies, eg. a ship which has lost power may have to anchor.

Design considerations for drill cuttings armour

The armour layers proposed in *Section 6.2* should be able to withstand the impacts of dropped anchors. Consideration of bearing capacity failures under the loads imposed by vertically falling anchors of up to 15 tonnes weight shows that penetration will be resisted by a rock or coarse gravel layer 2m thick. However, this would immediately be followed by dragging of the anchor that would cause damage. While it is generally understood that dragged anchors do not penetrate rock armour beyond a depth equivalent to one or two stones, they will inevitably displace some stones as they are dragged over the surface. Cumulative damage may occur over the long term that could lead to the exposure of the cuttings.

Far more significant damage may occur during a single event due to the effects of anchor chains attached to anchors which are dropped near to the covered pile. If a large vessel, such as a bulk ore carrier, suffers a malfunction that prevents control of the vessel, emergency stopping procedures will be implemented. The anchor will be lowered to the seabed, followed by a long length of heavy chain. The anchor will not immediately hold fast in the seabed but will be dragged along the surface in an erratic, twisting motion until the vessel has slowed sufficiently for the fluke of the anchor to hold without being pulled free by the momentum of the vessel. During this period, the anchor chain will be twisted and dragged across the sea floor. Some 400 metres of chain might be involved, whose links could be over a metre in length. The twisting chain will be capable of cutting through rock armour of substantial size, revealing the underlying material and possibly even reaching the cuttings themselves.

The risk of occurrence of such an event is extremely low. The cuttings piles in the UK Northern and central sectors are distributed over an area of approximately 60,000 km^2. Assuming that emergency anchoring events may occur randomly over the entire area, the chances of one event directly impacting an average 10,000 m^2 cuttings pile are approximately one in six million. The risks increase in the case of, for example, a bulk carrier dragging its anchor during an emergency stop but are still very low.

Thus, although it might reasonably be expected that anchor impacts by vessels involved in the oil industry can be avoided, and that emergency anchoring by other vessels in the future will be very rare events, it is not possible to guarantee that damage of the cover will not occur at some time in the future due to anchor impacts. As was concluded in the context of trawling impacts, this has implications for future monitoring and maintenance requirements.

6.5 IMPACTS DURING DECOMMISSIONING AND STRUCTURE COLLAPSE

Decommissioning scenarios and their implications for protection of the cuttings piles and disturbance of cuttings during and following decommissioning are reviewed below.

6.5.1 Full removal of fixed steel structure

Cuttings Displacement During Decommissioning

Full removal will in most, if not all, cases require disruption of the cuttings pile to enable the structure to be removed. Cuttings will need to be displaced from around the template and legs and from some low-level horizontal members. Because most cuttings piles are at their highest within or very close to the structure, this may entail displacement of a substantial proportion of the total volume. On completion of decommissioning, the remaining cuttings pile may have a very complex morphology. The problems of covering without triggering instability may be exacerbated and construction may be relatively complex and costly. In such cases, and subject to the development of appropriate cuttings recovery systems (including disposal), it may be more economic to simply recover all of the cuttings prior to decommissioning.

Impact of Objects Dropped During Decommissioning

It is possible that parts of structures will be dropped during the removal operation and there is a significant probability that any such drop will be sufficient to cause release of cuttings materials into the water column, even if the pile has already been covered with the initial layer of sand.

A representative item that might be dropped is a length of drains caisson and, for consideration of the energy at impact, it is postulated that a 1000mm diameter, 20mm thick steel tubular 10m in length falls a distance of 80m through water. Generally when a tube falls through water it travels a greater distance laterally than vertically. However, caissons and other appurtenances below platforms are usually restrained by vertical guides which would limit the lateral movement. For a tubular falling vertically, it can be shown that the impact energy could be up to 700 kJ. It can further be shown by considering the bearing pressure due to the impacting object on an armoured cover that, with this energy, vertical penetration would be in the order of 1.5m. It is unlikely that a sand cover would provide sufficient protection unless it were very thick.

6.5.2 Partial removal of fixed steel structure

Cuttings Displacement During Decommissioning

With the partial removal of a fixed steel jacket there will be no need to remove any of the structure below the top of the footings. The top of footings is the top of the foundation piles for most Central and Northern North Sea jackets, and the distance of the top of the piles from the sea bed is a function of the size and location of the platform and the design of the connection between the foundations piles and the jacket. The older platforms (generally those installed before 1982) have 'long' pile sleeves and the footings may be up to 60m above seabed, while the younger platforms have 'short' sleeves. The largest North Sea fixed steel platform, BP's Magnus, has short pile sleeves which extend only some 30m above seabed. These elevations are well above the levels of the cuttings piles and it will therefore only be necessary to remove or displace cuttings to access the ends of pipelines and umbilicals.

Impact of Objects Dropped During Decommissioning

The risk of disruption of the cuttings piles during partial removal are the same as those considered in the previous section and the same dropped caisson can be considered representative of a reasonably likely event.

Impacts of Post-Decommissioning Collapse

After the partial removal of the jacket the remains of the jacket will degrade due to corrosion. If the jacket's sacrificial anodes are left in place, degradation may not become significant for 1 or 2 centuries. Jacket corrosion systems were typically designed for an expected field life of 20 to 40 years. However, the design assumptions have been shown by experience to be conservative for North Sea conditions. Less than 10% usage of sacrificial anodes has been observed after 10 years in service, hence the expectation of 1 to 2 centuries before significant degradation commences. Alternatively it may be decided, or there may be a regulatory requirement, to remove the anodes. In such cases degradation will begin earlier. In both cases, there will be a gradual loss of strength of the remaining structure which will lead to progressive collapse.

The lower parts of the jacket are the strongest as they are designed to carry both the weight of the topsides and the moments caused by the wave, current and wind forces on the upper parts of the structure. At the lower levels, the legs carry both these components while the diagonal braces between the legs are designed to transmit the horizontal loads to the base of the jacket and into the foundation piles. The remains of the jacket will experience a gradual reduction in strength but they will also be subject to an immediate reduction in applied forces due to the removal of the topsides and much of the substructure. Thus some 70 to 85% of the strength will have to be lost before there will be any likelihood of collapse. It is therefore not unreasonable to expect the jacket remains to last between, say, 100 years (CP system removed at decommissioning) and 500 years (CP system left in place).

Notwithstanding the time it may take, collapse is inevitable, and pieces of the structure will fall onto any covered drill cuttings that lie below. The pieces that will collapse fall into three categories:

1) Horizontal bracings between the jacket legs below the top of the footings,
2) Vertical diagonal bracings between the jacket legs, and
3) Legs above the top of the drill cuttings piles.

Tubular falling horizontally:

This is the most likely scenario, since most objects falling through water will assume a near horizontal position early in the fall as a result of any eccentricity in the drag resistance or in the weight. It can be shown that the impact energy of the typical tubular considered, falling horizontally, would be of the order of 50 kJ. It can further be shown, by considering the bearing pressure due to the object impacting on the armour, that the vertical penetration would be in the order of 0.1m.

Diagonal brace and leg sections falling horizontally:

For the brace impacting the covering horizontally, it can be shown that the impact energy would be in the order of 130kJ; and for the leg section impacting the covering horizontally, it would be in the order of 700kJ. It can further be shown, by considering the bearing pressure due to the object impacting on the armour, that the vertical penetration would be in the order of 0.1m in each case.

Diagonal brace and leg sections falling vertically:

This is unlikely to occur but is considered to illustrate the potential impacts of an extreme event. For the brace impacting the covering vertically, with the same impact energy as above, it can be shown by considering the bearing pressure, that the vertical penetration into the granular covering would be in the order of 1.6m. For the leg section impacting the covering vertically, with the same impact energy as above, it can be shown that the vertical penetration into the granular covering would be in the order of 1.0m.

Thus if the covering has a rock armouring of not less than 1m thick, and a gravel layer of not less than 1m thick over the sand covering, the falling objects which have been considered would not penetrate further than these two layers, and in practice would probably not penetrate beyond the rock armouring.

6.5.3 Full removal of concrete platform

Cuttings Displacement During Decommissioning

As noted in Section 2, drill cuttings may lie on top of and against the bases of concrete platforms. Cuttings lying on top of the concrete base will have to be removed before the base is raised in order to reduce weight. In addition, it is likely to be necessary to remove them to prevent dispersion during recovery of the base. It is also likely that much of the external pile would have to be removed because it will, in most cases, be necessary to gain access to the whole of the perimeter of the bottom of the structures to inject water under the platforms to release them from the sea bed. In many cases, this may entail the removal of 80-90% of the cuttings (Galbraith, pers comm). In view of this, in situ covering of cuttings piles is not considered to be a viable option.

6.5.4 Partial removal of concrete platform

Cuttings Displacement During Decommissioning

Partial removal of a concrete platform means the removal of the upper part of the structure so as to leave at least 55m of unobstructed water below sea level. For structures with multiple legs, removal might be to as low as just above the top of the base structure (for some structures these may be the same depth).

Where partial removal option is chosen it is feasible to consider covering the cuttings lying on the seabed adjacent to the base prior to removal of the upper part of the structure, although disconnection of pipelines obstructed by drill cuttings would have to be accommodated. The seabed piles could probably be fully armoured before the main decommissioning work starts on the topsides and legs. However, in light of considerations of stability and the need to re-contour piles with steep slopes (refer to *Section 7*), it is most unlikely that covering the cuttings lying on top of the base will be a practical option and these cuttings are assumed to be removed.

Impact of Objects Dropped During Decommissioning

There is a considerable weight in the section of concrete to be removed. For the smaller diameter multiple-leg platforms, the part of each leg to be removed will weigh more than 6,000 tonnes in air, with far greater weights to be removed from the single-leg platforms. Prior to the removal of the upper part of the legs, the topsides will have to be removed. The risks to the drill cuttings are therefore from the removal and lifting operations of both topsides and of the upper concrete. The probability of dropping an item during heavy lift operations generally decreases as the weights increase, due to the increased amount of engineering and planning taken for the operation. The greatest risk to the drill cuttings pile is therefore from items dropped during the topsides removal and the situation will be similar to that described in *Section 6.5.1*, ie. a length of drains caisson impacting the armoured cover with an energy of up to 700 kJ with vertical penetration into the armour of the order of 1.5m.

Impacts of Post-Decommissioning Collapse

After decommissioning the concrete structure it will eventually deteriorate, and ultimately collapse. There are very little data available concerning concrete fatigue for real structures and none for an offshore production platform. However, the timescale for commencement of collapse could be centuries for the remains of the legs and, possibly, millennia for the caisson structures. Notwithstanding the time taken, total collapse will eventually occur and the resulting impact energies will be too great for any credible armouring of a cuttings pile to withstand.

6.5.5 Concrete platform left in place

Cuttings Displacement During Decommissioning

Although there are relatively few concrete platforms (about 24 in the North Sea compared with over 400 steel platforms), it is likely that several of the oldest will be left in place. The current OsPar agreement allows these platforms to remain if a derogation is obtained. During decommissioning, the topsides of any platforms left in place will be removed, as required by regulation, and during this activity there is a risk of dropped objects, just as for the other structures. Such dropped objects can be assumed to be characterised like objects dropped from the other structures (*Section 6.2*).

Impacts of Post-Decommissioning Collapse

After decommissioning, the structure will deteriorate and ultimately collapse. There are very little data available concerning concrete fatigue for real structures and none for an offshore production platform. However, the timescale for commencement of collapse could be centuries for the legs and, possibly, millennia for the caisson structures. Notwithstanding the time taken, total collapse will eventually occur and the resulting impact energies will be too great for credible armouring of a drill cuttings pile to withstand.

6.6 CONCLUSIONS

It is concluded that it is possible to provide rock armour protection to the cuttings piles which will be sufficient to withstand extreme storm events. However, repeated trawling and anchoring impacts and collapse of the structure which remains following partial decommissioning represent a threat to pile covers:

- while covers can be designed and constructed to withstand the impacts of trawling in the short and, perhaps, medium term, it is not practical to provide guaranteed indefinite protection to repeated trawling impacts which could be accidental or deliberate;

- similarly, while it is possible to construct covers which will not suffer 'terminal' damage from isolated anchoring impacts, it is not possible to protect against repeated impacts; although these are expected to be extremely rare, they could, in the long term, remove some of the cover protection;

- covers designed to withstand extreme environmental impacts offer some resistance to objects falling from any remaining structures but some damage will be inevitable (and possibly cumulative) and, for large structural members, damage may be severe.

All of these imply that, unless it can be shown that the contamination of the cuttings degrades in the short to medium term to the extent that they no longer pose a threat to the environment, there will be a requirement for monitoring and maintenance of the covered piles which may extend far into the future.

Consideration of peak particle velocities under storm conditions shows that there is a risk of damage to covers during construction. Storm events are likely to result in partial loss of an unprotected sand layer and, depending on the site characteristics and the severity of the storm, an unprotected gravel layer. As far as is practical, construction should be scheduled during a period in which conditions are forecast to be relatively benign and the sand and gravel layers should be armoured as soon as possible after placement.

In addition, it has been noted that during structural decommissioning, it is likely that some cuttings will need to be removed in order to gain access to buried parts of the structure (and to pipelines and other services). In some cases, especially full decommissioning, a substantial proportion of the total volume may have to be removed and the logic of covering the remainder, rather than completing the recovery process, is questionable, if only on economic grounds.

Although not strictly within the scope of this Study, it is also noted that partial removal of structures presents a risk (broadly similar to that posed by sunken wrecks) to fishing vessels because of the potential for snagging of fishing gear on the remains of the structure.

7 GEOTECHNICAL ASPECTS OF COVER DESIGN

7.1 INTRODUCTION

The engineering characteristics of the cuttings will depend upon their composition and stress/strain history. The latter will in turn be dictated by the manner of their deposition and the loads and displacements which have occurred since the piles were formed. The probable existing state of the cuttings piles is examined below followed by a review of their long-term behaviour after covering and the geotechnical factors affecting design and construction of the cover.

7.2 EXISTING STATE OF CUTTINGS PILES

7.2.1 Deposition mechanisms

Compared with most natural sedimentation processes, which take place in a geological timescale, the cuttings piles have accumulated rapidly - over about three decades, at most. The materials forming the piles have been discharged from points located at varying heights above the seabed and at a varying rates. From consideration of estimated pile heights, areas and volumes (*Section 2.3.3*) the existing piles appear to be conical or sub-conical in shape, but with greatly varying side slopes. Depending upon currents at the time of discharge, cuttings will have dispersed laterally from the discharge points.

A cohesionless soil (eg. clean sand) deposited onto a rigid horizontal surface from a single discharge point in still water will tend to form a conical mound with slopes at an angle roughly equal to its angle of internal friction, ϕ. However, the greater the discharge height, relative to the height of the mound, the greater will tend to be the lateral dispersion of particles due to advection and diffusion during descent. Small volumes of cohesionless soil discharged from large heights may not therefore approach this limiting condition. Similarly, the more rapid the deposition, the greater will be the effect of disturbance and, assuming other factors equal, the flatter and less regular will be the slopes.

The form of cuttings piles composed largely of silt and clay particles is less predictable but they would generally be expected to produce flatter profiles than clean sands. However, should displacements occur following deposition and initial consolidation, they might be more capable of sustaining steeper local slopes in the short-term. This is a general characteristic of cohesive soils that would apply to cuttings containing a significant proportion of clay-size particles and may explain the locally steep slopes reported in some piles.

These general comments are based on saturated soils with water as the pore fluid. When other fluids completely or partially fill the voids between particles, as is the case for some the piles, soil behaviour may be significantly different. This will depend upon the nature and concentration of the fluids.

It should be noted that the process of deposition involves shear displacements and flow of the cuttings, which is technically a failure condition. Where such movements are confined to thin layers accumulating on the slopes of existing cuttings, this is of limited practical significance. However, in view of the irregular nature of the deposition processes, much larger volumes of cuttings may have suffered shear failure after their initial placement. This is considered further in the following section.

7.2.2 Influences since deposition

It was concluded in *Section 2.3.3* that the slopes of existing cuttings piles are typically concave with maximum angles (possibly as steep as 35-40^o) occurring in the upper part of the slope and much smaller angles (perhaps only 1^o to 10^o) occurring lower down. For many small piles, the restraint provided by parts of the structure may be an important factor governing the steeper upper slopes but this is less likely for larger piles.

Following deposition, the cuttings will have been subjected to external forces from waves, currents and possibly earthquakes. Due to the nature of the constituents, it is also possible that the internal stresses within piles may have been modified by chemical processes. Many of these influences would have been concurrent. For the purpose of discussion they are considered separately below.

Stresses and strains within the cuttings piles may be sub-divided into those associated with failure states (when the induced shear stresses reach the shear strength of the material) and those occurring without failure. The former include liquefaction brought about typically by repetitive or cyclic stressing, leading to complete loss of shear strength. Since the cuttings piles are thought to consist largely of saturated soft or loose soils, failure from any cause would probably be accompanied by large displacements and soil flows.

The shear strength and stiffness of a soil depend primarily upon the effective stresses (effective stress is the difference between the total stress on the soil and the pore water pressure) between the soil particles, both in their existing state and all previous states, i.e. the 'stress history' of the soil. Influences which bring about changes in total stress and pore pressure, and hence in effective stress, must therefore be closely examined to see whether they might lead to instability of the soil mass or, alternatively, increase its strength and stiffness.

Self-Weight

The sedimentation of fine-grained soils is a complex process involving the transition from a very low density suspension of particles to the development of a structure of particles of slightly higher density capable of resisting shear stress. Thereafter, any soil element below the surface of a pile will be progressively stressed by the weight of further cuttings deposited above. Provided failure does not occur, there will be a gradual increase in strength and stiffness as the pore fluid is squeezed out. This process is described as consolidation and, in the absence of any other applied loadings, the soil mass is said to undergo 'self-weight consolidation'.

The time required for consolidation depends primarily on the permeability and structure of the soil. Since individual cuttings samples have generally been found to contain a wide range of particle sizes, it appears that segregation of particle sizes during deposition has been limited. The properties of the cuttings materials after deposition are therefore assumed to have no preferential direction, i.e. they are assumed to be isotropic.

Provided that shear failure has not played a significant part in the development of a pile, the magnitude of consolidation depends on the magnitude of applied stresses and the compressibility of the cuttings materials. Compressibility is indicated by the coefficient of volume compressibility, m_v and is highly stress-dependent. At low stress levels soon after deposition, particles will be in a loose condition with a correspondingly high compressibility. As consolidation takes place and the soil becomes denser, compressibility will decrease.

The rate of consolidation (ie. the rate at which the pore fluid is expelled) is a function of the drainage path length (essentially the thickness of the layer being consolidated) and the permeability of the soil structure. The rate of consolidation is indicated by the coefficient of consolidation, c_v, which is also dependent on stress level. In their initially loose condition, soils will usually consolidate more rapidly than when they have been compressed into a denser state.

The magnitudes and rates of consolidation at any point within a cuttings pile will therefore be greatest during the early stages of its development and will decrease as the soil density increases. However, since the cuttings piles are conical or sub-conical in shape, stress conditions will vary with both time and location. Beneath the centre of a pile, the vertical total stress will be greatest and will have been applied for the longest time. The stress will decrease towards the edge of the pile. At points near the circumference, loading will have begun much later than at centre and the duration of loading will therefore be shorter. However, the length of the drainage path will also be shorter because the thickness of cuttings will be less.

If the approximate rate of deposition is known, and if the pile is assumed to maintain the shape of a perfect cone with constant slope angle, the approximate vertical total stress at any location at any time can be calculated. In practice, this is not solely dependent upon the vertical height of material above that point; the size and shape of the pile also have influence.

Consider, for example, a conical cuttings pile of volume 15,000m^3 formed at a side slope of 20^o by deposition at a constant rate over a period of say 10 years. After 5 years, its height at centre would be 9.8m, 79% of its final height of 12.4m. However, at this stage no cuttings will yet have been deposited over the outer 37% of its final plan area. The rate of increase in height of the pile at any point will also vary greatly with time and location. After 3 months the pile height would be increasing at a rate of about 13mm per day whereas after 10 years this would have reduced to just 1mm per day.

This analysis relates to an idealised cuttings pile with the form of a perfect cone but it illustrates that non-uniform stress conditions will have developed in the piles during deposition. Since the soil properties are dependent upon applied stress magnitude and the period over which it is applied, it is clear that there will be considerable variation in properties from point to point within individual piles.

The degree of consolidation at any stage will also depend on the nature of the underlying seabed soils. If they are more permeable than the cuttings materials, the excess pore water pressures caused by loading would be able to dissipate by flow to this lower boundary, as well as to the upper surface of the pile. Cuttings piles may be located on a wide variety of seabed soils ranging from very permeable sands to highly impermeable stiff clay. For the purposes of this assessment, it is assumed that the piles rest on impermeable materials and that drainage is one-way to the free surface.

Consolidation is considered complete when excess pore water pressures generated by a given loading have dissipated and hydrostatic conditions have been restored. In piles comprising permeable cuttings with a low fines content, this condition may have been approached, but other influences must be considered (see below). However, as most cuttings piles appear to contain a significant proportion of fines, and the coefficients of consolidation measured during the recent tests indicate that pore pressures probably remain generally in excess of hydrostatic even though several years may have passed since cuttings deposition ceased.

Fine-grained soils in which excess pore pressures have dissipated are said to be 'normally-consolidated' and their strength and stiffness usually increase with depth. Soils that exhibit excess pore pressures are described as 'under-consolidated' deposits. There are no reliable indications of the existing strengths of the cuttings pile materials but, as the available data suggest that most cuttings piles are likely to comprise underconsolidated materials, there is no strong reason to believe that shear strengths will increase with depth in the normally-consolidated manner.

Provided there are no shear failures, the volume of pore fluid expelled as a pile accumulates will be governed by the consolidation process. If failure occurred under the weight of additional cuttings, or due to external forces, there would be a sudden increase in pore pressures and a decrease in the effective stress between particles. The benefits of previous consolidation would be lost and consolidation would re-commence only after equilibrium had been regained.

Bearing Failure

Where the seabed is relatively stiff, e.g. boulder clay or dense sand, displacements will be confined largely to the cuttings materials. Soft sediments on the seabed (even if present only in thin layers) would greatly increase the likelihood of failure involving both the cuttings and the foundation soils. Bearing failure may have occurred repeatedly for some of the piles. Information on strata conditions below original seabed level would be required to confirm whether this is a likely mechanism at any particular site.

The initial low self-weight of the cuttings will enable a greater thickness to be placed over any soft sediments before failure takes place but it is unlikely that a pile height of more than about 5 metres could have been achieved without failure on a soft substrate. Such failures would have been accompanied by large displacements or soil flow and resulting in further loss of strength as pore water pressures in both the cuttings and the seabed soils rapidly increased. After failure, some of the excess pressure would have dissipated gradually but since subsequent discharges would be deposited onto weakened material, the probability of further failures would be high. The net effect of this mechanism would be lateral spreading of the cuttings and relatively shallow pile slopes.

The seabed conditions at the locations of the cuttings piles have not been investigated as part of this study, but it is quite possible that some of the piles currently exhibiting flat overall slopes have been formed in this way, particularly the larger piles which would have imposed higher bearing pressures on the seabed. Additional forces from waves, and possibly earthquakes, may also have come into play and could have provided the immediate trigger for failure.

Waves

Waves cause differential loading on the sea floor, changing the total stresses and pore pressures within the seabed soils and the cuttings piles. Wave-induced bottom pressures depend largely on wave height, wavelength and water depth. In relatively permeable soils, the induced pore pressures may also depend upon wave period. Wave regimes have been assessed in *Section 5* for two conditions: an average condition assumed to correspond to a 1-year event and an extreme condition based on a 1:500-year storm event. For the purpose of geotechnical assessment, it is assumed that all piles, during formation and after completion, have suffered wave conditions at least equivalent to a 1-year event.

The magnitudes of wave-induced pressures are calculated on the assumption that the sea floor is rigid and close to horizontal. The compressibility of the cuttings and the irregular, sloping form of the cuttings piles are not taken into account. Since the induced pressures are non-uniform, they cause additional shear stresses in the soils as well as increasing pore water pressures, again in a non-uniform manner. These effects are greatest where the ratio of water depth to wavelength, h/L, is least. In 'deepwater' conditions ($h/L > 0.5$), magnitudes are very small.

The stresses below sea bed level can be estimated on the assumption that the soil is an elastic continuum. In the case of the cuttings piles, which are of limited height and irregular profile, this is a crude approximation and may underestimate the true stresses. Nevertheless, given the many other uncertainties, this is considered adequate for present purposes.

For wave loadings equivalent to an annual event, the maximum increase in shear stress ranges from about 6 kPa in the Moray Firth (eg, Beatrice with h = 46m) to 2 kPa in the Central sector (h = 140m) where h/L values are less than 0.5. These shear stress increments are small but significant when compared with the likely shear strengths of the soft and loose soils comprising the cuttings piles. The increase is negligible in the Northern sector where h = 170m and h/L = 0.52.

The wave-induced pressures also cause a transient increase in pore pressure. In view of the very short loading time (determined by the wave period) compared with the permeability of most of the cuttings soils, it is assumed that no drainage occurs. The increase in pore pressure at seabed level is therefore assumed equal to the applied stress but decreases with depth as the total stress increments also decrease. As well as the transient pore pressures arising from the 1-year wave, repeated cyclic loading of contractive soils will tend to generate residual pore pressures in the soils in excess of hydrostatic pressure. These will be in addition to those due to incomplete consolidation. This phenomenon is known to occur in all soil types and causes a reduction in shear strength compared with the same soil subjected only to static loading

To date, the influence of waves will have been to increase shear stresses in the cuttings piles and decrease their effective shear strength. The stress increases are additive to those arising from gravitational forces, i.e. from the sloping profile of the piles. In some cases, the shear strength of the cuttings materials will have been adequate to resist these forces; in others (especially where the slopes were initially steep) failure may have occurred.

Earthquakes

Unlike the effect of waves, the effect of earthquakes on soils below the sea bed does not attenuate with water depth. Horizontal travelling waves in the bedrock propagate vertically into the overlying deposits causing cyclic shear stresses which may induce a significant increase in pore pressure and decrease in soil strength.

The horizontal accelerations resulting from earthquakes are commonly represented as an equivalent static horizontal body force which, in sloping ground, introduces an additional destabilising force. Earthquakes thus have the potential to increase the disturbing forces on the cuttings pile slopes and simultaneously decrease the resisting forces. Where relatively steep submerged slopes exist at low factors of safety, even low intensity earthquakes can provide the stimulus for collapse and consequent flow of liquefied soil. The extent to which the cuttings piles have been modified by earthquakes since their initial deposition is unknown but it seems possible that some may have been affected at some stage of their development.

7.2.3 Existing stability

The flow of pore fluid from cuttings piles due to self-weight consolidation will have been taking place from the beginning of deposition. Where the cuttings consist mainly of fine-grained particles, such as those tested during this Study, it is almost certain that this process is continuing, at least within the cuttings in the higher central parts of the piles. Since shear strength is controlled by effective stress (the difference between total stress and pore water pressure), the residual excess pore pressures from incomplete consolidation will have an important affect on stability. They will vary from pile to pile depending on the nature of the cuttings, their rates of deposition, strata conditions below seabed and the effects of waves, currents and possibly earthquakes. Instability at any earlier stage of pile development will also have had a radical effect.

Existing pile profiles therefore do not provide a reliable indication of the state of stability of the piles. It would be wrong to conclude that piles with flatter slopes are inherently more stable than those with steeper slopes, or that covering and protecting the existing slopes with thin granular layers will ensure their long-term stability. Whilst the gravitational shear stresses will be greatest where slope gradients are steep, resistance to failure is dependent on the effective shear strength of the soils and hence on the unknown excess pore water pressures. Only where piles can be shown to consist mainly of coarser particles would it be reasonable to conclude that self-weight consolidation is substantially complete.

Until information is available on the *in-situ* state of individual cuttings piles, it would be prudent to assume that some cuttings piles, in their present condition, have only limited safety margins against instability.

7.3 FUTURE STATE OF CUTTINGS PILES

This section reviews the geotechnical limitations of the covering solution in terms of stability during and after construction, the potential for the cover to contain leachate expelled during future consolidation and potential leachate expulsion rates.

7.3.1 Stability

The existing state of the cuttings piles is a product of their physical characteristics, the manner of their deposition and subsequent loadings due to self-weight consolidation, surface waves and earthquakes and the effects of possible soil failure and mass displacements. The same factors must be taken into account when considering design for long-term conditions. In addition, the effect of construction of the covers must be taken into account.

A major design requirement is that the cuttings piles in their protected state must remain stable. Failures due to any of the factors identified in the previous section could destroy the continuity of the layered protection system, expose the cuttings and increase the release of contaminants to the environment.

Covering Layers

The covers will impose loads on the cuttings piles, increasing the total stresses and pore water pressures and thus giving rise to increased shear stresses. The imposed loads will cause further consolidation of the cuttings piles. The increased stresses also have the potential to cause instability because they will be applied rapidly relative to the slow consolidation process at work in the cuttings materials.

The pore pressure increments resulting from placement of the covers will be additive to those already existing within the cuttings piles. In general, the greater the thickness of cover, the greater will be the increase in pore pressure; and the less uniform the applied loading, the greater will be the additional induced shear stress. For piles that are currently stable (ie. at a factor of safety sufficient to resist future worst-case loadings due to waves and earthquakes), the covers should ideally consist of uniform layers of sufficient thickness merely to satisfy their required functions. In practice, far greater thicknesses may be required. This is because of the constraints (identified in Section 6) to placing armour protection on steep slopes, leading to the conclusion that the maximum design slope for armouring should be 18^o.

Many piles have markedly concave slopes, the upper parts of which are steeper than 18^o. In order to reduce these slopes to 18^o or less it will be necessary either to regrade the slope by placing a large thickness of sand in the concave section or to remove the top of the pile prior to covering.

Placing a sufficient thickness of sand to provide a linear slope from head to toe of the pile may require very large volumes. For example, in the case of the 'large conical pile with high shape factor' (based on the dimensions of the Fulmar A pile with a shape factor of about 9) illustrated in *Figure 2.12A*, the volume of granular materials would need to be about twice the volume of the cuttings, and the thickness of the cover would vary between 2-3m at head and toe of the slope and 7-8m at a distance of about 20m from the centre.

Such large increases in loading, allied to the highly non-uniform pore pressures induced in the cuttings, could cause instability unless placement was at a controlled slow rate or in a series of lifts of limited height. In this context, 'slow' means placement over a period of many months, possibly years. Staged covering in a series of lifts may involve extended waiting periods between each lift. This approach is likely to be impracticable and/or prohibitively expensive.

The viability of the alternative approach of removing the peak of the cuttings pile depends on the successful development of cuttings excavation equipment and the identification of an appropriate disposal solution. Both Phases I and II of this JIP have broadly concluded that cuttings excavation and recovery to the surface is feasible, but the disposal solution remains a key issue. In this case, one option may be to simply displace the cuttings to a lower part of the slope rather than to transport them to the surface for disposal. However, this process would result in the slurrification of the cuttings and it would be extremely difficult to:

a) place the cuttings on even the very shallow lower slopes of a pile in such a manner that prevents them from flowing onto the adjacent seabed; and
b) cover them after they have been placed.

Given these difficulties, and recognising that the merits of recovery to the surface have yet to evaluated in the context of the other options for dealing with cuttings piles, our initial view is that recovery to the surface is likely to be the easiest option if the peaks of piles have to be removed before covering. An advantage of this approach is that relatively small volumes will be involved and the disposal problem may not be great.

Waves

Wave-induced pressures on the sea floor due to storm waves in the permanent condition have been estimated for an extreme 500-year event. Relative to the depth of the cuttings piles below sea level, the additional thickness of the covers will not be significant and, whilst the profiles of the piles will be modified, the method of estimating these pressures does not take into account the different shapes or slope angles of the piles. However, their potential effect will be greater for more steeply sloping piles.

The arising forces are assumed to depend only on the nature of the waves. The five example site scenarios considered in *Section 6* (ie. Moray Firth, Central Sector shallow and deep water, and Northern Sector shallow and deep water) have h/L (water depth/wavelength) values of between 0.16 and 0.32 for the 500-year event. The maximum increase in shear stress ranges from about 10 kPa in the Moray Firth (h = 46m) to 2 kPa in the Northern Sector (h = 170m). Again, it is assumed that no drainage occurs during wave loading and that the transient increase in pore pressure at seabed level is equal to the increase in applied total stress.

It was noted in *Section 7.2.2* that repeated cyclic loading of contractive soils leads to a build-up of residual pore pressure in excess of hydrostatic and hence a reduced soil strength. An extreme wave event that occurs when pore pressures are already high due to slow consolidation under self-weight and/or the additional stresses from placed covering layers, increases the likelihood of soil failure.

Earthquakes

It was concluded in *Section 7.2.2* that earthquakes may have been a factor in the development of some piles. The longer the design life of the piles and covering system, the greater will be the probability of larger magnitude earthquakes occurring and the greater the potential for instability or damage. The looser the soils the more susceptible they will be to strength reduction or possible liquefaction. Clearly, the greater the factor of safety against instability under normal loadings, the greater will be the marginal shear strength available to resist any additional earthquake forces.

It is noted that consideration of earthquake forces should not be restricted only to the cuttings but will also be relevant to the design of the cover itself. Large volumes of loose granular soils are prone to liquefaction under earthquake loadings particularly if slope angles are steep. Thus, covers which include a great thickness of sand placed to create a regular finished profile may be susceptible to earthquake failure. These factors will need to be considered in further detail in the design of each pile.

Long-Term Consolidation

Assuming that the cover materials can be placed to the design profiles without causing instability, the shear strength of the cuttings will increase with time as the excess pore water pressures due to self-weight and additional static load from the covering materials gradually dissipate. As consolidation takes place, the ability of the protected piles to resist external forces without failure will also increase.

7.3.2 Accommodation of leachate in cover

Consideration has been given to the potential for granular covering layers to accommodate the contaminated pore fluid expelled from the cuttings piles due to continuing self-weight consolidation and the additional consolidation due to the load of the covers themselves. Since little information is available on the *in-situ* state of the cuttings piles (as distinct from their composition and disturbed properties), estimates of expelled pore fluid volumes (and the rates of expulsion derived in *Section 7.3.3*) have been based on the following broad assumptions:

1. The cuttings piles consist of fine-grained soils having properties similar to those measured for the three samples tested as part of this Study (*Section 2.5.3*). The results of earlier investigations suggest that some piles contain a greater proportion of coarser particles than the recent samples from Beryl and Ekofisk which appear to have particle size distributions near the fine end of the overall range so far observed. This should be borne in mind when applying the results elsewhere.

2. The density of cuttings close to the surface of piles is similar to the average density measured in the laboratory on disturbed test specimens prepared under a small static load.

3. Granular coverings will be placed in uniform layers, thus causing uniform stress increases. In practice, placement of the granular layers will not be uniform; the moving front of fill material will tend to displace a wave of loose disturbed cuttings at the leading edge. Also, the flow of pore fluid from the covered cuttings will take place in the direction of maximum pore pressure gradient and is therefore likely to have a down-slope component.

4. The flow of pore fluid will take place vertically. This assumes that the cuttings are isotropic or that any discrete layers of relatively high permeability are aligned approximately parallel to the pile slope and do not 'daylight' in the slope. The latter assumption is logical in the light of the manner in

which the cuttings piles have been formed but may not always be the case in practice. If such layers do exist, and daylight in the slope, self weight consolidation may be more advanced than is assumed here, additional consolidation under the load of the cover may occur faster than would otherwise be the case and pore water expulsion may be focused at the points where these layers daylight.

5. No flow of pore fluid or leaching out of contaminants from the upper levels of the cuttings piles has taken place since deposition, other than that due to the consolidation process. If the assumption of vertical drainage under (4) above is incorrect, it is possible that the self-weight consolidation estimated here would be an underestimate leading to a possible overestimate of the pore water expulsion under the load of the cover.

Self-weight consolidation will have begun almost immediately after initial placement of the cuttings. In their loose state, the rate of consolidation, and hence the rate of expulsion of pore fluid, will have been fairly rapid. However, the recent tests indicate that rates decrease greatly as the cuttings become denser. Once consolidated under the increased overburden pressure 2m or 3m of additional cuttings, the measured rates decrease by a factor of about 10; and a further decrease, by a factor of about 5, is indicated for stress levels equivalent to a pile height of about 10m.

If these results are typical of the more fine-grained cuttings materials, it is unlikely that the self-weight consolidation will have been substantially completed for piles with heights greater than about 5m. However, whilst significant excess pore pressures may exist within the larger cuttings piles, they are of more relevance to stability. The rate of deposition of large piles (in terms of their height increase with time) is small, and the rate of expulsion of pore fluid will be correspondingly low.

Estimated magnitudes of self-weight consolidation for pile heights up to 15m are shown in *Figure 7.1*. The curves begin at 0.5m as the top 0.5m of cuttings are assumed to remain in a semi-liquid state. The estimated total consolidation resulting from the combination of self-weight consolidation and the consolidation due to the load of granular covering materials of between 2m and 8m thickness are also shown. All curves show the magnitudes after 100% consolidation irrespective of the time required to achieve this condition.

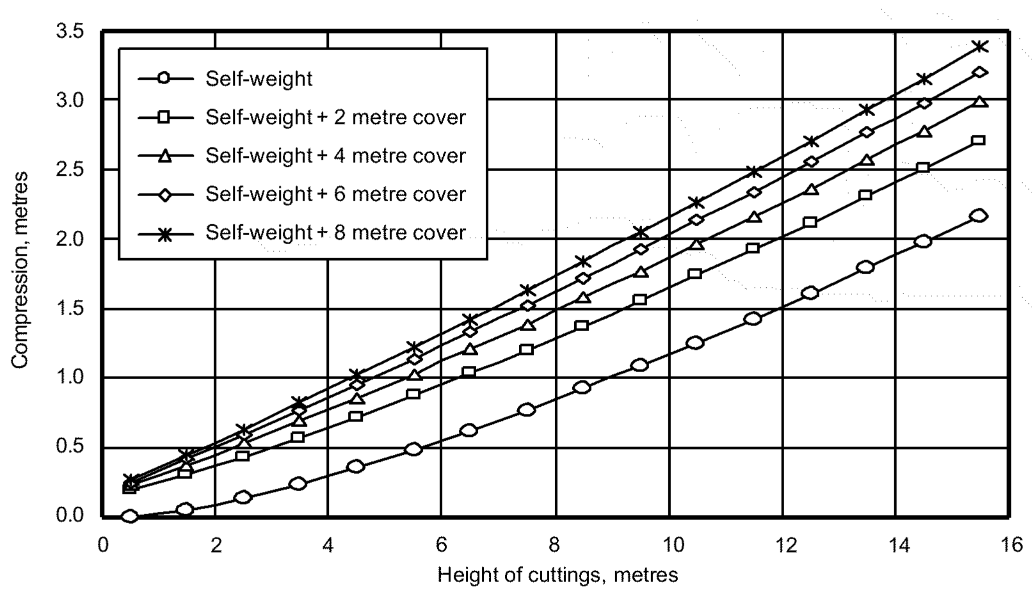


Figure 7.1 Estimated magnitudes of consolidation of covered cuttings piles

Based on the test undertaken during this study, indicative error margins of about ±20% might apply to the curves shown on Figure 7.1. The full extent of pile-to-pile variation is, at present, speculative.

If covers are to be designed to accommodate all of the pore fluid expelled from the cuttings piles, their void volume must be at least equal the volume of pore fluid yet to be expelled due to continuing self-weight consolidation plus the additional volume expelled due to the weight of the coverings themselves. The greater the thickness of cover the greater will be the additional loading and hence the additional compression but the volume of voids available to accommodate the expelled leachate will also be greater. For example, the additional compression resulting from 8m of granular fill is about twice that resulting from 2m of fill whereas the void volume is directly proportional to the thickness and will therefore be about four times greater. Hence, the critical case for absorption will be where the granular covering is thinnest.

The additional compression induced by a 2m-thick cover varies from 0.25m for a 1m thickness of cuttings (including the assumed semi-liquid layer) to just over 0.5m for a 15m cuttings thickness. This is because the 15m pile, after assumed full self-weight consolidation, is substantially less compressible. In practice, it is most unlikely that full consolidation will have been achieved for a 15m pile and therefore the covers would need to accommodate an indeterminate additional volume.

The highest pile with the thinnest granular covering therefore represents the worst case in terms of the volume required for absorption. For 100% self-weight consolidation, 0.5 m^3 of expelled pore fluid (per square metre) would need to be accommodated within a minimum thickness of say 2m of granular coverings. The required minimum void ratio (ie. the ratio of void volume to solids volume) would therefore be 0.33. Since a clean uniform sand would be expected to have a void ratio of at least 0.4, even in its densest state, this requirement could easily be satisfied.

In practice, for a medium-dense condition, the void ratio would probably be nearer 0.6 or 0.7. Therefore, where self-weight consolidation is incomplete, this would provide a margin for a further 300mm of continuing consolidation and thus a further 0.3 m^3 of expelled water. Even so, for a 15m high pile this represents only 15% of total self-weight compression and, based on the test data, it seems unlikely that piles of this height will have attained an 85% degree of consolidation prior to placement of the coverings.

In the light of the examination of the implications of contaminated pore water release presented in Section 8, it is arguable whether the granular covering layers need to be designed to accommodate all the pore fluid which would be expelled from the cuttings over a theoretically infinite time. However, if this were the case a thickness of 2m might be insufficient for piles with heights greater than about 10m.

7.3.3 Potential Expulsion of Leachate

Expulsion through completed cover due to consolidation

Where existing cuttings piles are continuing to consolidate under self-weight, current rates of consolidation will be slow. The placement of covering layers will be rapid and will cause an initially rapid expulsion of pore fluid. Since consolidation rates decrease greatly with increasing stress, it will be the upper levels of the cuttings piles that dictate this rate of expulsion.

Figure 7.2 shows the compression of a 1m high cuttings pile against time after placement (assumed to be instantaneous) of a 2m layer of sand, and *Figure 7.3* shows the corresponding rate of expulsion of pore fluid in litres per hour per square metre of cuttings surface area. The rate of expulsion decreases sharply with time. The relatively high initial rate in the first few days after placement would not be significantly different for piles of greater height, nor for a greater weight of covering materials. In practice, because of the time taken to place the sand, the maximum rates of expulsion are likely to be lower than those calculated.

However, the expelled leachate will not be released to the water column but will be accommodated within the sand. Assuming a conservative void ratio of 0.4, each square metre of a 2-metre thick sand layer could accommodate approximately 570 litres of leachate. This is three times the volume of leachate that will be expelled from the cuttings during the six weeks following sand placement, by which time expulsion will have reduced to an extremely low rate.

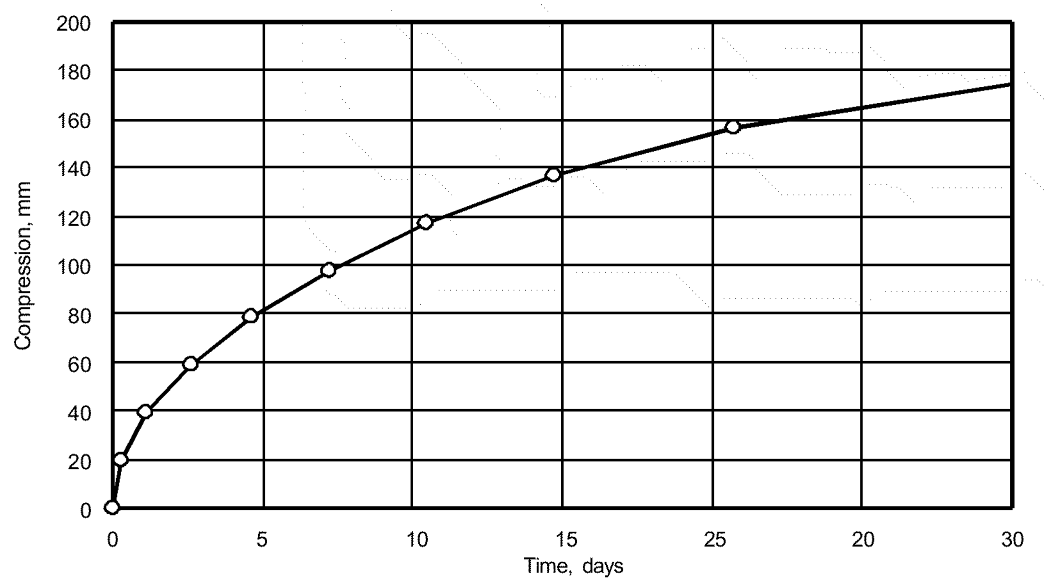


Figure 7.2. Compression against time for 1m cuttings after placing 2m of granular coverings

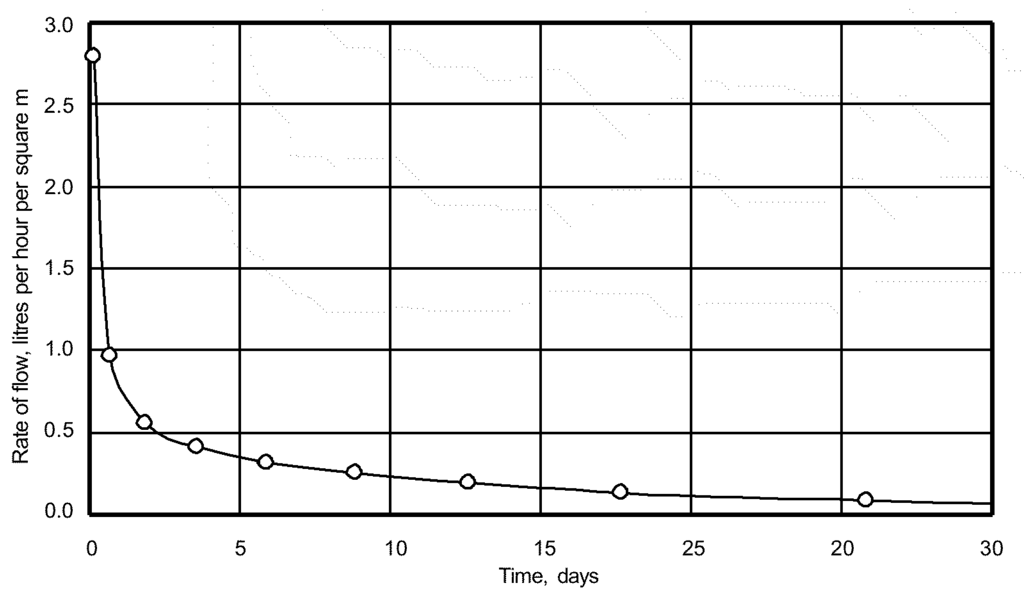


Figure 7.3 Rate of flow of pore fluid from 1m cuttings after placing 2m of granular coverings

On this basis, losses to the water column due simply to the expulsion of water during consolidation are unlikely to occur because the sand layer will be able to accommodate the water with a comfortable margin of safety. The exception to this is the case of a cuttings pile higher than 10 metres with a sand layer which is only 2 metres thick. Losses in this situation would occur at a very slow rate because the sand layer would only become filled with leachate in the very late stages of consolidation when expulsion rates will be extremely slow compared with the rates during construction.

Expulsion during construction

Some pore water will inevitably be released at the advancing front of the sand layer as it is placed and there will be some disturbance of the cuttings as the initial layer of sand is placed. It is difficult to estimate, in a rigorous manner, the rate of release that would be caused by these mechanisms but a reasonable approach might be to assume that the uppermost 0.25m of cuttings are disturbed to such an extent that they give up all of the pore water as the initial layer of sand is placed.

If this layer is assumed to have a water content of 100% (noting that the highest reported water content is 69% - refer to *Section 2.5.4*) and a particle specific gravity of 2.7, the volume of pore water that would be released would be approximately 180 litres/m^2 of disturbed area. The rate of placement of the sand cover will depend on the vessel that is used but is most unlikely to exceed about 4,000 m^3/day. A 2-metre thick sand layer would be placed at a maximum rate of about 2,000 m^2/day. At this placement rate, pore water would be released to the water column at about 4.2 litres/sec.

The duration of the release will depend on the area of the pile being covered and the details of the methods and equipment used by the contractor. The largest reported pile area is that of Thistle A, at 24,400 m^2. It would take approximately 12 days to place the sand layer on a pile of this size assuming continuous production at maximum efficiency. Most piles could be covered more quickly, the smaller ones in perhaps 3 or 4 days. In practice, placement is likely to be discontinuous and maximum efficiency may not be maintained at all times. Leachate losses to the water column are thus likely to be discontinuous and, on average, slower than those estimated here.

Expulsion due to the effects of waves

Losses could occur following placement of the sand layer as a result of the 'pumping' action due to wave-induced differential pressures in the cover. As a wave advances across a covered pile, it will generate a hydraulic gradient within the permeable sand layer. During the passage of the first half of the wave, the hydraulic gradient will induce an approximately vertical flow of pore water into the sand layer. As the second half of the wave passes, the flow will be reversed. In order to assess the potential magnitude of this effect, the worst-case scenario of a 1:500 year event in the shallow Moray Firth has been reviewed.

The 12-second waves during an event of this magnitude would generate alternating positive and negative hydraulic gradients of 0.075 in a 2-metre thick sand layer. Assuming a sand permeability of $1*10^{-4}$ m/sec, approximately 0.03 litres of pore water will be expelled from each square metre of sand cover and a similar amount will flow back in during the passage of a single wave. This exchange process will involve only the pore water very close to the surface of the sand layer. The outflow of 0.03 litres is equivalent to the void volume of the uppermost 0.1mm of the sand layer.

As noted above, it is highly unlikely that a 2-metre sand layer would become saturated with pure leachate from the cuttings except in the case of the very largest piles (which could, of course, receive a thicker covering of sand if necessary). However, if it is assumed that the sand layer is saturated with leachate, it becomes important to establish what will happen during the course of an event comprising numerous waves.

During each wave passage, 0.03 litres of pore water is lost from the sand and is replaced 6 seconds later. The water that flows back into the sand could be the full-strength leachate that has just been expelled or it could be fresh, uncontaminated seawater or a mixture of both. It is most unlikely that all of the expelled leachate will immediately be drawn back into the sand without any dilution but it is clear that, if this occurred, there would be no overall loss of leachate to the water column. It is equally unlikely that all of the expelled leachate would be immediately swept away from the surface of the sand and replaced entirely by clean seawater. The water that flows back into the sand layer is therefore likely to comprise a diluted leachate.

The diluted leachate that passes back into the sand layer could mix with the pure leachate that has risen to the surface or displace it back down into the sand layer. In practice, there is likely to be some mixing but, during successive wave passages, the process will be repeated with the result that both the leachate expelled from the sand and the pore water in the upper part of the sand layer will become progressively more dilute.

This process will ensure that, after only a few minutes, the rate of loss of (pure) leachate will become almost unmeasurable, irrespective of the degree of dilution that is assumed to occur above and within the sand layer. For example, if 0.03 litres of water is expelled during each wave and immediately diluted by 50%, and the same volume is drawn back into the pile to mix 50:50 with undiluted leachate, the loss of (pure leachate) during the passage of the first wave will have been 15 litres per 1,000 m^2 of cover. During the passage of the 50th wave, 10 minutes later, the loss of (pure) leachate will have reduced to $1.1*10^{-5}$ litres per 1,000 m^2.

The estimated losses for various rates of dilution (of the water returned to the cover) during the first ten minutes of a 1:500 year event are summarised in *Table 7.1*. The losses are expressed in terms of full-strength leachate.

Table 7.1 Estimated leachate losses due to pumping action in a fully leachate-saturated sand layer during a 1:500 wave event (Moray Firth).

Dilution (%) of previously expelled water returned into cover	*Loss during first wave passage, litres per 1,000 m² of cover*	*Loss during 50th wave passage, litres per 1,000 m² of cover*
0 (ie. all leachate returned)	0	0
25	7.5	0.01
50	15	$1.13 * 10^{-5}$
75	22.5	$2.24 * 10^{-9}$
Infinite (ie. no leachate returned)	30	$5.33 * 10^{-14}$

Different figures can of course be derived using different assumptions about dilution and mixing rates and the permeability of the sand but the estimated rate of loss of leachate rapidly becomes negligible.

These figures are based on an very conservative scenario that assumes:

1) the sand layer is fully saturated with pure leachate from the cuttings;
2) there has been no loss of leachate from the cover between the time of completion and the start of the 1:500 year event.

In fact, the pumping action will commence as soon as sand is placed but the initial loss rates will be smaller than those described above because construction will take place during periods of relative calm and the covered pile will almost certainly be subject to a long period of relatively benign wave climate before a 1:500 year event occurs. In addition, induced hydraulic gradient will lower for the vast majority of piles because they lie in much deeper water than the Moray Firth.

It is therefore concluded that, although a pumping action will operate within the cover, it will not give rise to significant rates of leachate release to the water column. The only situation that would invalidate this conclusion would be the case of a pile that:

1) had consolidated after cover construction to the extent that the entire sand layer had been filled with leachate, and,
2) was continuing to consolidate at rate which ensured that losses through the cover were matched by the rate of supply of leachate from the cuttings.

This situation is most unlikely to arise and would represent a failure of the cover design process leading to the specification of a sand layer thickness inappropriate to the morphological and geotechnical characteristics of the pile in question.

7.3.4 Implications of Reading University test data

The results of the geotechnical tests undertaken at Reading and received in the final stages of this Study do not have any major impact on the conclusion drawn in *Section 7.3*. The implication of higher values of c_v is that the pore water expulsion during self-weight consolidation prior to covering would have been greater than assumed here, thus reducing the amount to be expelled under the weight of the cover. While the rate of expulsion immediately after cover construction might be greater than assumed here, the pore water would pass into the sand layer which should be sufficiently thick to accommodate all expelled water. Loss of leachate to the water column due to the pumping effect of waves is dependent on the properties of the sand layer rather than the cuttings themselves.

7.4 OUTLINE COVER DESIGNS

On the basis of the factors described above, four outline designs have been developed to cover a range of existing profiles and geotechnical assumptions. These are based on the normalised pile morphologies developed in *Section 2.4* and illustrated in *Figures 2.12* and *2.13*.

7.4.1 Design criteria

It is evident from the foregoing review that it is not possible to develop at this stage cover designs which, with a high degree of confidence, can be constructed without giving rise to instability. This is a simple function of the uncertainty concerning the *in situ* properties of drill cuttings. However, a number of preliminary design guidelines have emerged from the consideration of pile morphologies, armouring requirements, construction methods and geotechnical constraints (as they are presently perceived). These have been incorporated in the outline designs presented here which are used in *Section 11* as the basis for indicative construction costs and energy budgets. The preliminary guidelines are:

1) 'reasonable' protection of the cuttings (including the sand layer containing contaminated pore water) can be provided by a 2-metre layer of coarse granular material (taken to be the gravel and armour stone layers combined);

2) consideration of the available methods of material placement suggests that the minimum nominal thickness of each layer of covering material should be about 1 metre; a 3-layer cover will thus be at least 3.0 metres thick and the combined gravel and stone protective components will meet or exceed the 2 metres required for 'reasonable' protection;

3) difficulties of accurate deep-water placement of coarse material and the limitations of fall pipe vessels in combination with the size of armour stone required on slopes of different angles suggests that finished slopes should not exceed 1:3 (18^{o});

4) consideration of stability suggests that existing steep cuttings slopes may need to be 'built-out' during covering as they not be able to support the loads imposed by a cover either during or after construction; for the purposes of this study it is assumed that pronounced concavities in existing cuttings piles (which suggest a fine cuttings composition) will need to filled so that the maximum finished slope is 1:5 (11^{o});

5) for conical piles (suggestive of a relatively coarse composition), the maximum finished design slope is assumed to be determined by the slope limitations for armouring, ie. 18^{o}.

With respect to item (4), it is recognised that, subject to the identification of appropriate excavation and disposal techniques, the alternative to building out large slope concavities would be to remove the tops of the piles.

7.4.2 Large pile with high shape factor

This pile is based on the (normalised) dimensions of the Fulmar pile and is illustrated in *Figure 2.12A*. The available evidence suggests that piles of this type are composed of cuttings at least as fine as those tested recently. Progressive shear failure may have been a significant mechanism in developing the existing markedly concave profile. Rates of consolidation will have been slow, with the probability that pore pressures within the central parts of the pile remain high.

The combined effect of pile height and pronounced concavity presents a design problem for the granular coverings. Simply following the existing profile would result in a very steep inner slope where the pile in most vulnerable, whereas a single linear slope from head to toe would require a very large volume of material and would cause a very large increase in stress on the cuttings.

Figure 7.4 shows a compromise broken-slope profile which might prove suitable. However, the maximum fill thickness of 7m is still a substantial increase in load, especially as it would be placed in a relatively short period. Whilst steepening the inner slope reduces the applied load intensity near the inner slope 'toe', it also increases shear stresses within the central parts of the pile where pore pressures are probably still high. This delicate balance would have to be evaluated on a case by case basis when conditions at individual piles have been investigated.

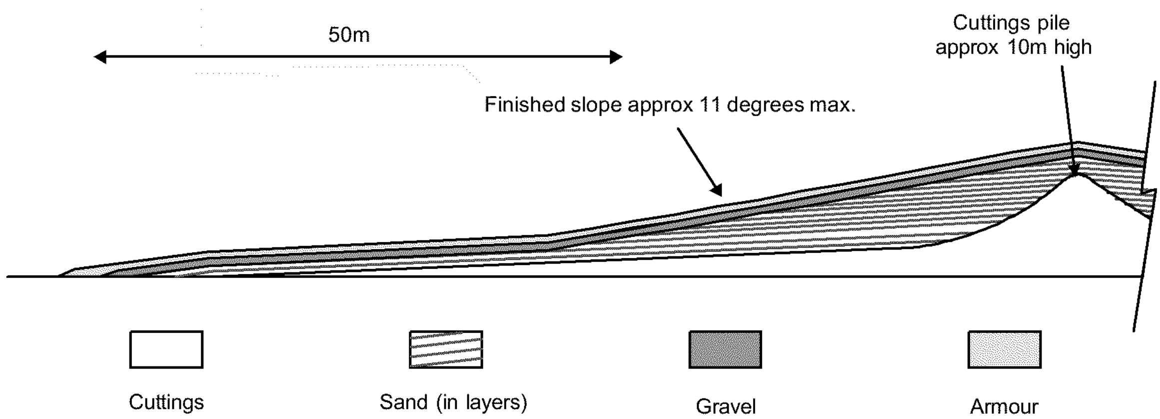


Figure 7.4 ***Outline design for cover on a large pile with high shape factor***

As noted earlier, an alternative solution might be to remove the upper 2m or 3m from the top of the pile, thus reducing the maximum cover thickness from 7m to 5m and decreasing the inner slope angle from 11° to 8°. Clearly, both effects increase the factors of safety. In view of the concavity of the pile profile, the volume to be removed would be relatively small.

7.4.3 Large conical pile (shape factor ≈ 3) with steep slopes

This pile is based on the (normalised) dimensions of the Murchison pile and is illustrated in *Figure 2.12B*. Whilst no geotechnical information is available, this pile profile suggests conditions superior to those inferred for the previous example. Its shape appears to be nearly conical and for this height of pile, the 26° near-linear side slopes strongly suggest that the cuttings are relatively coarse and may consist mainly of sand with a low or negligible fines content.

Like most granular slopes built up in this fashion, the existing factor of safety is likely to be fairly small (probably less than 1.2). The finished profile, shown at 18° in *Figure 7.5*, should provide a reasonable increase sufficient to cater for the permanent worst-case loadings.

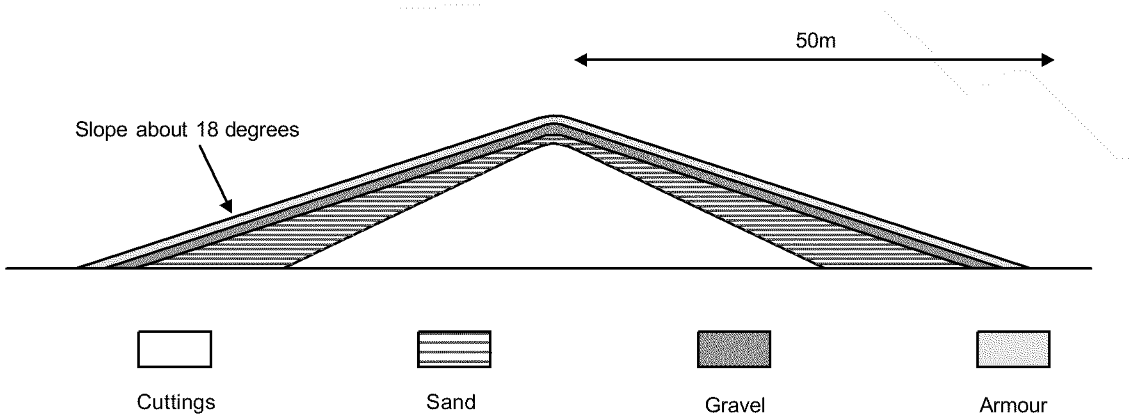


Figure 7.5 ***Outline design for cover on a large conical pile with steep slopes***

However, even though such piles may consist very largely of sand and/or gravel, there is no reason to believe that the cuttings are in other than a loose state. Therefore, in spite of their apparent stability, pending site investigation it would be prudent to assume that some piles may remain susceptible to degradation or, in the worst case, liquefaction, due to future worst-case dynamic loadings.

7.4.4 Large conical pile (shape factor ≈ 3) with shallow slopes

This pile is based on the (normalised) dimensions of the NW Hutton pile and is illustrated in *Figure 2.12C*. Whilst this profile is again nearly conical, the side slopes are at about 5°, very much flatter than those of the previous example. In the specific case of NW Hutton, the cuttings appear to consist largely of silt and this may be a factor. However, it is also possible that the flat slopes result from a lateral spreading of particles due to weak seabed sediments or other local cause. The finished profile (*Figure 7.6*) is obtained by placing a minimum thickness of granular covering materials.

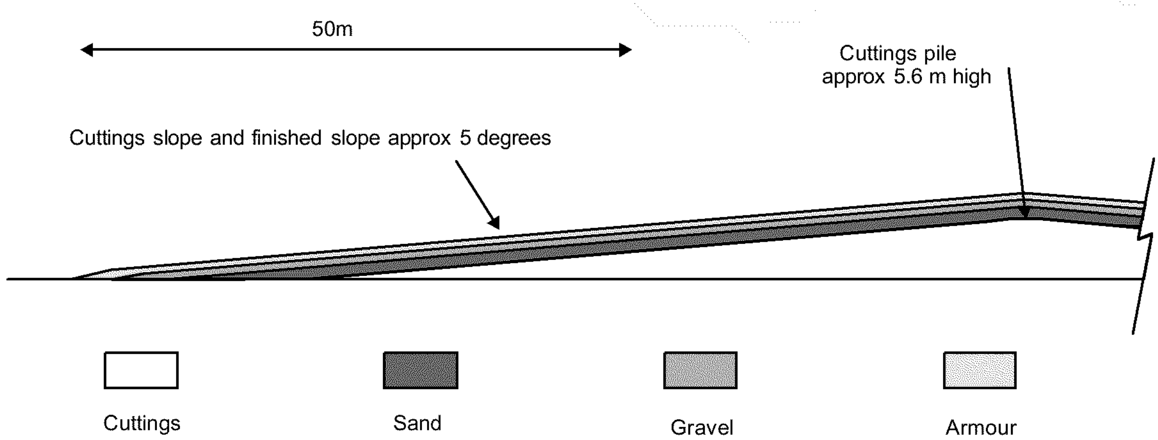


Figure 7.6 Outline design for cover on a large conical pile with shallow slopes

7.4.5 Small piles (<2,500 m^3)

A cover design for a typical small pile is shown in *Figure 7.7*, based on the normalised dimensions of the Osprey and Kingfisher piles (*Figure 2.13*).

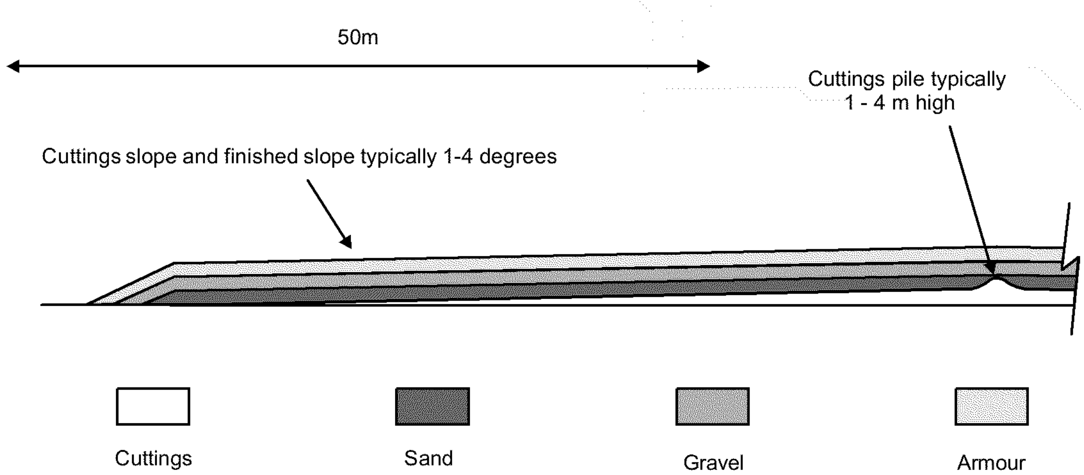


Figure 7.7 Outline design for cover on a small pile

As noted in *Section 2.4.4*, the shape factors of small piles (ie. < 2,500 m^3) are unlikely to have a significant bearing on cover design. This is because, irrespective of the shape factor, the data on measured piles suggests that the pinnacles are very small and typically only a few metres (3-10) in diameter. It would be very difficult to place materials at great depths with sufficient precision to build-out the steep slopes of these pinnacles. It is therefore assumed that the small volume of material contained within the pinnacles is either removed or, more likely, simply allowed to collapse under the load of the cover as it is placed. This is unlikely to give rise to a great release of cuttings to the water column or to complicate the covering operation because of the small volumes involved.

7.5 FURTHER GEOTECHNICAL INVESTIGATION

The available geotechnical data are limited and, in some cases, are of doubtful reliability or value. The additional laboratory tests undertaken as part of this Study on disturbed samples from Beryl and Ekofisk are considered reliable but constitute only an indicative basis for cover design. Most importantly, there are no reliable indications of the existing *in-situ* state of the cuttings, especially their density, shear strength and stiffness, and pore water pressures.

It is clear that the adoption of a covering solution must be preceded by additional detailed geotechnical investigations. These investigations must include the substrate on which the piles rest as this may have an important bearing on pile stability and is relevant to the assessment of the degree of cuttings consolidation and the release of leachate prior to covering.

In view of the water depths and the difficulty of obtaining 'undisturbed' tube samples of either the cuttings or the underlying soils, the most suitable method of obtaining the necessary data is likely to be the cone penetrometer operated in 'seabed mode'. Sampling will of course be necessary to supplement the penetrometer data and to assist in its interpretation and all possible precautions should be taken to ensure that they are as little disturbed as possible. The recommendations given in the introduction to *Section 2.5*, concerning adherence to recognised geotechnical standards and maximising the amount of data obtained from samples, should be applied.

7.6 CONCLUSIONS

Consideration of the geotechnical aspects of covering gives rise to concern about the stability of cuttings piles and the manner in which covers may need to be constructed in order to prevent instability during and after construction. Some of this concern flows from the paucity of geotechnical data but the data which are available, in combination with a consideration of the processes of cuttings pile formation, suggests that it would be imprudent to assume that all cuttings piles are sufficiently stable to accommodate the proposed coverings without some form of modification.

It is clearly the case that detailed geotechnical investigations will need to be undertaken at each site before covers are designed and designs should take into consideration the geotechnical properties of the underlying soils and allow for earthquake loadings. It should be recognised that, in some cases, that it may not be practical to construct a satisfactory cover.

It appears likely that it will be necessary to 'build out' existing steep cuttings slopes in order to minimise stability problems. This will have to be done by varying the thickness of the sand layer and will require careful placement techniques. This, in combination with consideration of the armouring requirements and placement methods, leads to the tentative conclusion that the maximum design slope should be about 18° for cuttings piles with 'favourable' geotechnical properties and 11° (possibly less) for piles with 'less favourable' characteristics.

An alternative approach would be to remove the top of the cuttings pile in order to reduce the thickness of cover which must be placed to achieve the required gradients and thus reduce the loading on the underlying cuttings. This would make cover construction easier but depends on the identification of appropriate cuttings removal and disposal methods. The approach may significantly reduce the cost of cover construction but the cost saving will be at least partly offset by the cost of removing the top of the pile.

The consolidation test data obtained during this study indicate that it will be possible to accommodate within a sand layer two metres thick the leachate that will be expelled from the majority of the cuttings piles, in the long term, due to consolidation. Slightly thicker sand layers may be required in the case of very high cuttings piles. Thinner layers may be sufficient for others. However, the requirement for total accommodation of leachate is dependent on the magnitude of the impacts which may arise from leachate release. This is considered in detail in the following section.

Estimates of leachate release have been made which suggest that releases through a completed cover due to the pumping action set up by waves would be quite insignificant as long as the cover was adequately designed. More significant releases are likely to occur during cover construction as a result of disturbance when placing the initial layers of sand. A preliminary estimate of about 4.2 litres per second has been derived based on the assumption that the uppermost 0.25 m of the cuttings will be sufficiently disturbed to release all of the contaminated pore water that they contain.

The results of the University of Reading's recent geotechnical tests do not significantly change any of the conclusions set out here except that they suggest consolidation under self-weight (of the Beryl and Ekofisk piles) may be more advanced than has been assumed and that the piles may therefore be more stable.

8 CONTAMINANT LOSS THROUGH PILE COVERS

8.1 INTRODUCTION

As the cover materials are placed, they will impose a load on the cuttings pile which will consolidate. Contaminated pore water will be expelled from the cuttings and pass into the cover, displacing the relatively clean seawater contained in the pore spaces of the cover materials. Even if the absorbent sand layer is sufficiently thick to accommodate all of the expelled pore water, the permeability of the cover means that, in time, contaminated leachate is likely to be released. This section of the report reviews the implications of leachate release based on analyses of leachate obtained from the cuttings samples from Beryl A and Ekofisk and on ecotoxicological tests undertaken over a wide range of dilutions.

In the early stages of North Sea development, diesel oil was used as the only oil-based fluid. In the early 1980s, diesel was replaced by mineral oils of lower toxicity. The Ekofisk field has been regularly monitored since 1973 and approximately 500,000 tonnes of drill cuttings have been discharged (Dames & Moore 1999)[1]. Most of the drilling has been done using synthetic base oils but approximately 1,000 tonnes of diesel have been discharged with the cuttings. However the Ekofisk samples are considered to represent an example of low oil based cuttings by comparison to the Beryl A cuttings.

8.2 SAMPLE COLLECTION AND HANDLING

Approximately 50kg of cuttings were sampled from the cuttings pile at the Beryl A and Ekofisk 2/4 A platforms. Cuttings samples were collected using a large box corer. The cuttings sample from Beryl A was collected from the Southeast edge of the primary cuttings pile at a distance of 60 metres from the Southeast corner of the platform. The primary cuttings pile associated with the Ekofisk 2/4 platform is orientated on a North East South West line. Two samples were collected from the Ekofisk cuttings pile approximately 10 metres from the platform. One sample was collected from the South West side of the platform and one from the North East (Ekofisk samples 1 and 2 respectively).

The cuttings material from the box cores was transferred to stainless steel containers and transported to the CEFAS laboratory in Burnham-on-Crouch. Upon arrival, a sub-sample of cuttings (approximately 10 kg) was removed from each of the three samples using a stainless steel scoop and transferred to plastic buckets for the geotechnical tests described in *Section 2.5.3*. A day after the cuttings had been sampled for the geotechnical tests, the water that backfilled the space remaining after removal of the cuttings sub-sample, was collected for analysis. Although not a standard way to collect interstitial water, a large sample volume was required quickly and it was considered that the sample would contain concentrations of contaminants from the cuttings that were representative of the levels normally present in the interstitial water.

8.3 TOXICITY TESTING

8.3.1 Methods

Cuttings water samples for the toxicity test were collected from the sample container in a solvent-cleaned glass beaker so as to prevent contamination from plasticisers, such as phthalates, present in all plastics. In order to test the toxicity of the cuttings water samples, each was diluted using a clean filtered seawater sample, collected from the River Crouch, to produce a dilution series. In this way the amount of dilution required to remove the toxic effects could be determined. The dilution series included 100% clean seawater and 100% cuttings water and five dilutions containing 3, 6, 10, 33 and 56% cuttings water mixed with clean seawater.

[1] Dames & Moore, 1999. *Historical review of drill cuttings at the Greater Ekofisk field.* Dames & Moore Report for Phillips Petroleum. Dames & Moore: Stavanger, Norway. 16 pp. + apps

Crustaceans are abundant and important in the marine environment. Therefore in order to determine the toxicity of the cuttings water samples a common crustacean species, *Tisbe battaglai* (Higgins & Thiel 1988[1], Giere 1993[2]), that normally lives in the interstitial water surrounding marine sediment particles, was tested. Twenty organisms were placed in each dilution of the cuttings water and the numbers surviving in each dilution of cuttings water after 24 and 48 hours were recorded. In order to make a comparison between the toxicity of the cuttings water samples the dilution producing 50% mortality of the crustacean test species was calculated after 48 hours exposure for each cuttings water dilution series.

8.3.2 Toxicity results

The results of the toxicity tests are presented in Table 8.1.

Table 8.1 Percentage mortality of the crustacean Tisbe battaglai after exposure to a range of cuttings water dilutions.

% of cuttings water in dilution	*Percentage mortality after 24 and 48 hours (% @ 24 / % @ 48)*		
	Beryl A	*Ekofisk 1*	*Ekofisk 2*
100	0 / 0	15 / 30	100 / 100
56	15 / 15	5 / 5	25 / 100
33	0 / 0	10 / 10	15 / 25
10	5 / 5	5 / 10	5 / 5
5.6	0 / 0	0 / 0	0 / 0
3.3	0 / 0	5 / 5	0 / 0
0 (control)	0 / 0	0 / 0	0 / 0

Beryl A

There was 15% mortality of the crustacean *Tisbe*, after 24 hours exposure to a dilution containing 56% cuttings water from Beryl A. There was also 5% mortality in a dilution containing 10% cuttings water after the same exposure period. No deaths occurred in the other cuttings water dilutions so mortality could not be attributed to the toxicity of the sample but may have resulted from handling stress.

Ekofisk – Sample 1 - southwest side

After 24 hours exposure to a 100% cuttings water solution 15% mortality of *Tisbe* occurred. At a dilution containing 33% cuttings water10% mortality was observed. In dilutions containing 56, 10 and 3.3% cuttings water, 5% mortality occurred after 24 hours. After 48 hours exposure, the mortality increased to 30% in 100% cuttings water. In a dilution containing 33% cuttings water, mortality increased to 10% after 48 hours exposure, but remained the same in all other cuttings water dilutions.

Ekofisk – Sample 2 - northeast side

In the Ekofisk 2 sample, increasing mortality of Tisbe is associated with increasing concentrations of cuttings water. 100% mortality occurred after 24 hours exposure to 100% cuttings water.. In dilutions containing 56, 33 and 10% cuttings water, 25, 15 and 5% mortality respectively were recorded. After 48 hours exposure, mortality had increased at a further two concentrations; to 100% in a dilution containing 56% cuttings water and to 25% in a dilution containing 33% cuttings water. Based on these results the dilution that would produce 50% mortality of Tisbe after 48 hours was calculated as 32%.

[1] Higgins, R.P.& Thiel, H. (eds.), 1988. *Introduction to the study of meiofauna*. Smithsonian Institution Press, Washington, London 488 pp.
[2] Giere, O. 1993. *Meiobenthology - the microscopic fauna in aquatic sediments*. Springer-Verlag, Berlin, 328 pp.

8.3.3 Discussion of toxicity results

In the Beryl A sample, no clear relationship was observed between mortality and cuttings water dilution. Several organisms died during the test but it is thought likely that the mortality was due to handling stress.

Mortality of test organisms appeared only to be associated with cuttings water concentration in the Ekofisk samples, suggesting that some properties of the water were responsible for the mortality. The low dissolved oxygen concentration in Ekofisk sample 2 is likely to have stressed the organisms and therefore contributed to the high mortalities in this cuttings water dilution series. However, there was a high concentration of dissolved oxygen in Ekofisk sample 1 and yet 30% mortality was observed in the undiluted cuttings water, suggesting that other factors contributed to mortality. Ekofisk sample 2 produced the highest mortality. A standard value quoted in toxicity studies is the concentration (or percentage) of a toxic substance that produces 50% mortality of a test species after a defined exposure time. This standard value is termed the LC_{50}. For Ekofisk sample 2, the 48-hour LC_{50} was a dilution of 32% of cuttings water (*ie.* a 1:3 dilution).

8.4 CHEMICAL ANALYSIS

8.4.1 Analysis criteria and methods

The sample analysis methods are described in Appendix B to this report. A wide range of chemicals may be present in cuttings piles and may occur at concentrations that will have adverse effects on marine organisms. The selection of chemicals for analysis was made using two criteria:

1) chemicals shown in previous studies to be associated with cuttings material and which are sufficiently soluble to be present in solution at concentrations which are toxic to aquatic organisms;
2) chemicals for which there are established and cost effective methods of analysis.

Based on knowledge of the toxicity of the measured compounds and their concentration in the interstitial water, it is possible to determine whether the majority of the observed toxicity can be attributed to the measured compounds alone or whether other chemicals are also making a significant contribution.

A range of *hydrocarbons*, including a number of polycyclic aromatic hydrocarbons (PAHs), was analysed since some of these are sufficiently water-soluble and toxic that they may make a significant contribution to toxicity at the concentrations found in the interstitial water.

Several *metals* normally present in seawater at trace concentrations may be particularly concentrated in the cuttings piles as a result of the drilling activity and may then be accumulated by marine organisms and produce harmful effects. Those metals previously detected in cuttings samples and which are sufficiently soluble to be present at toxic concentrations were also selected for analysis in the cuttings water.

Historically, *nonylphenol ethoxylates* (NPEs) (a group of industrial chemicals with surfactant properties) were widely used as emulsifiers in drilling fluids and as cuttings cleaners and rig washes. Although the use of NPEs is now covered by a voluntary agreement intended to reduce their use offshore, their breakdown products become strongly associated with sediments and may only be partially degraded over a period of several months particularly in anoxic conditions (Ekelund et al., 1993[1]). Since one of the major breakdown products of NPEs, nonylphenol, has been demonstrated to have a range of harmful effects to aquatic organisms (Servos, 1999[2]) it was considered important to include this chemical and some of the other major breakdown products of NPEs in the present analysis. A related chemical, octylphenol, which is present in many surfactant formulations and possesses similar properties to nonylphenol was also measured.

[1] Ekelund R., Granmo A., Magnusson K., Berggren M., and Bergman A., 1993. *Biodegradadtion of 4-nonylphenol in seawater and sediment.* Environ. Pollut. **79** 59-61.
[2] Servos M. R., 1999. *Review of the Aquatic Toxicity, Estrogenic Responses and Bioaccumulation of Alkylphenols and Alkylphenol Polyethoxylates.* Water Qual. Res. J. Canada., **34**, 123-177.

8.4.2 Chemical analysis results

The results of the analyses are presented in *Tables 8.2* (hydrocarbons), *8.3* (trace metals) and *8.4* alkylphenolic chemicals.

Table 8.2 Range of hydrocarbons measured in the cuttings water samples. All figures in µgl⁻¹ with the exception of total hydrocarbons (THC) which is in mg/l⁻¹

Chemical determinand	*Beryl A*	*Ekofisk 1*	*Ekofisk 2*
n-alkanes	247	212	93
Sulphur	90	7.3	94
Naphthalene	67	67	86
C_1-Naphthalenes	105	109	110
C_2-Naphthalenes	<0.1	<0.1	<0.1
C_3-Naphthalenes	<0.1	<0.1	<0.1
Acenapthylene	<0.1	<0.1	<0.1
Acenapthene	<0.1	<0.1	<0.1
Fluorene	<0.1	<0.1	<0.1
Phenanthrene	<0.1	<0.1	40
Anthracene	<0.1	<0.1	8.7
C_1-Phenanthrenes/Anthracenes	<0.1	<0.1	<0.1
Fluoranthene	<0.1	<0.1	<0.1
Pyrene	<0.1	<0.1	<0.1
Chrysene	<0.1	<0.1	<0.1
1,2-Benzanthracene	<0.1	<0.1	<0.1
Benzo[b+j+k]fluoranthene	<0.1	<0.1	<0.1
Benzo[*e*]pyrene	<0.1	<0.1	<0.1
Benzo[*a*]pyrene	<0.1	<0.1	<0.1
Perylene	<0.1	<0.1	<0.1
Indeno[1,2,3-*cd*]pyrene	<0.1	<0.1	<0.1
Benzo[*ghi*]perylene	<0.1	<0.1	<0.1
Dibenz[*ah*]anthracene	<0.1	<0.1	<0.1
Total Hydrocarbon Content (as Forties crude oil equivalents).	0.2	<0.1	<0.1

Table 8.3 Concentration(in µgl⁻¹) of trace metals measured in the cuttings water samples.

Sample	*Manganese*	*Nickel*	*Copper*	*Zinc*	*Cadmium*
Beryl A	13.0	7.19	0.23	1.33	0.008
Ekofisk 1	178.2	5.80	1.40	0.74	0.083
Ekofisk 2	24.5	2.10	0.31	3.84	0.009
EQS value	Note 1	15	5	10	Note 2

Note 1: An EQS value for saltwater has not been derived since the bioavailability of manganese is likely to markedly reduced in seawater. However the maximum allowable concentration for freshwater is 300µg /l and the annual average 30 µg /l.
Note 2: An EQS value has not yet been established in the UK for cadmium

Table 8.4 Concentration of specific alkylphenolic chemicals measured in the cuttings water samples. All concentrations are expressed in µgl⁻¹.

Compound	*Beryl A*	*Ekofisk1*	*Ekofisk2*
Octylphenol	0.18	0.04	0.02
Nonylphenol	0.54	<0.1	0.26
Nonylphenol mono ethoxylate	1.80	0.21	0.43
Nonylphenol di-ethoxylate	<0.6	0.70	0.94

8.4.3 Discussion of chemical analysis results

Beryl A

Analysis using coupled gas chromatography/mass spectrometry (GC/MS) produced chromatograms in which different hydrocarbons could be identified and quantified. Total ion current chromatograms (TIC, the sum of all ions studied) were used to display the overall signal for extracted compounds and individual mass chromatograms were generated to separate the signals for specific compounds. An example Beryl chromatogram is shown in *Figure 8.1* where the TIC is shown with the mass chromatogram for 256 Daltons, corresponding to the molecular weight of the cyclic sulphur molecule S_8. There is a possibility that a proportion of this elemental sulphur derives from the oxidation of other sulphur compounds during sample handling, although this compound is frequently found in anoxic sediments. Although it may not be as toxic as some other inorganic and organic sulphur compounds, its toxicity to fish and their larvae has been demonstrated and so it is a compound of concern in the context of cuttings piles (Svenson et al., 1998[1]).

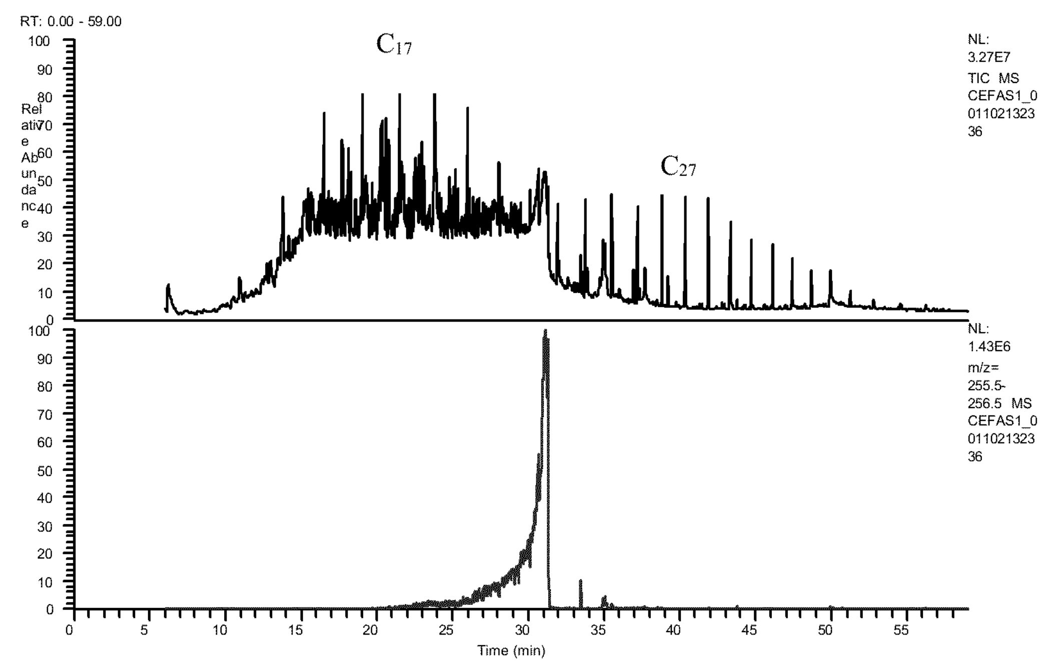


Figure 8.1 GCMS chromatogram for the Beryl A sample (upper plot - total ion current; lower plot, selected mass chromatogram for 256 Daltons showing presence of a broad peak due to sulphur).

This characteristic sulphur peak deriving from bacterial degradation of organic matter under anoxic conditions can be seen in the cuttings water chromatogram. The hydrocarbon analysis shows the presence of residues reflecting the use of both diesel based and low toxicity drilling muds, the latter mainly manufactured using de-aromatised kerosenes, plus a higher-boiling fraction which could derive from weathered crude oil. The boiling range of the dominant drilling mud base-oil components from both the leachate and the cuttings samples are very similar to that observed in a sediment sample collected in 1980, 2 miles from the Beryl A platform (Law & Blackman, 1980[2]). However, the sample from Beryl A was collected from the primary cuttings pile and shows less degradation of the *n*-alkane hydrocarbon components relative to that sample, as the cuttings pile probably has a low or zero oxygen concentration. Surface sediments at a greater distance from the platform are generally more oxygenated, allowing aerobic bacteria to multiply and to utilise the hydrocarbons as a carbon source, thereby degrading the *n*-alkanes whose peaks reduce in size as a result.

[1] Svenson, A., Viktor, T. and Remberger, M. (1998). *Toxicity of elemental sulfur in sediments.* Environ. Toxicol. Water Qual., **13**, 217-224.
[2] Law R.J., and Blackman, R.A.A., (1981). *Hydrocarbons in water and sediments from oil-producing areas of the North Sea.* ICES CM 1981/E: 16. 20pp.

The total hydrocarbon content of the Beryl A sample was 0.2 mg^{-1} and the concentration of *n*-alkanes was 247 μgl^{-1}. Naphthalene and C_1-naphthalenes were the only PAHs detected in the sample, at concentrations of 67 and 105 μgl^{-1} respectively. Nineteen other PAH compounds or compound groups analysed were at concentrations below the method detection limit of 0.1 μgl^{-1}.

Manganese was detected at the highest concentration (13 μgl^{-1}) of the five metals analysed. All of the metals were present at low concentrations. The concentration of alkylphenolic chemicals was also low relative to marine and coastal sites.

Ekofisk

The concentration of sulphur in the Ekofisk 1 water sample is estimated to be about 7 μgl^{-1}, while in the Ekofisk 2 sample it was approximately 90 μgl^{-1}. The isolation of the mass chromatogram for the ion at 57 Daltons (due to the butyl hydrocarbon chain fragment $C_4H_9^+$ in aliphatic hydrocarbons) provides a means of selectively enhancing the relative intensity of alkanes in the samples. Use of this technique reveals two small peak clusters within the chromatograms (*Figure 8.2*) for both Ekofisk samples that may be due to the use of low toxicity base-oils during the development phase of the field. The trace from Ekofisk sample 1 also shows two additional clusters which, based on the spectra, may be due to synthetic base-oils, possibly due to the use of ester or ether formulations during a later phase of development of the field.

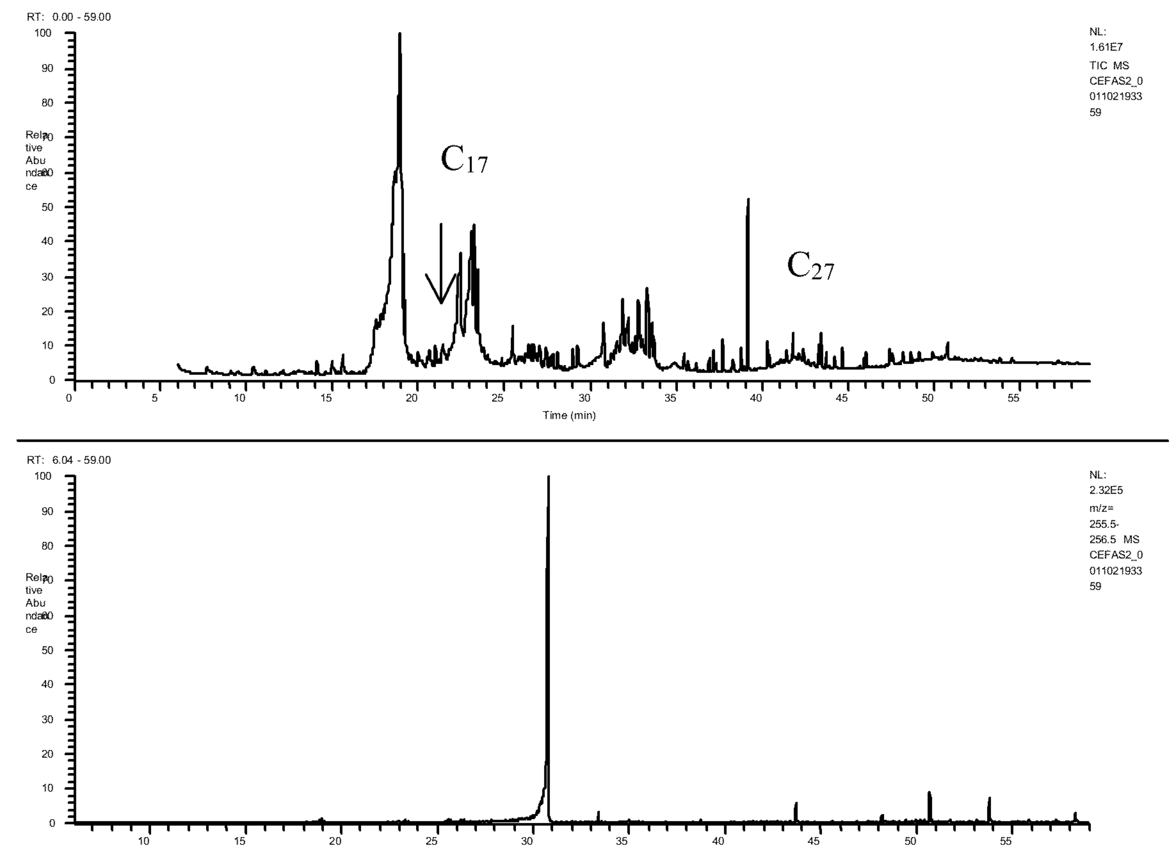


Figure 8.2. GCMS chromatogram for the Ekofisk 1 sample (plots as Figure 8.1).

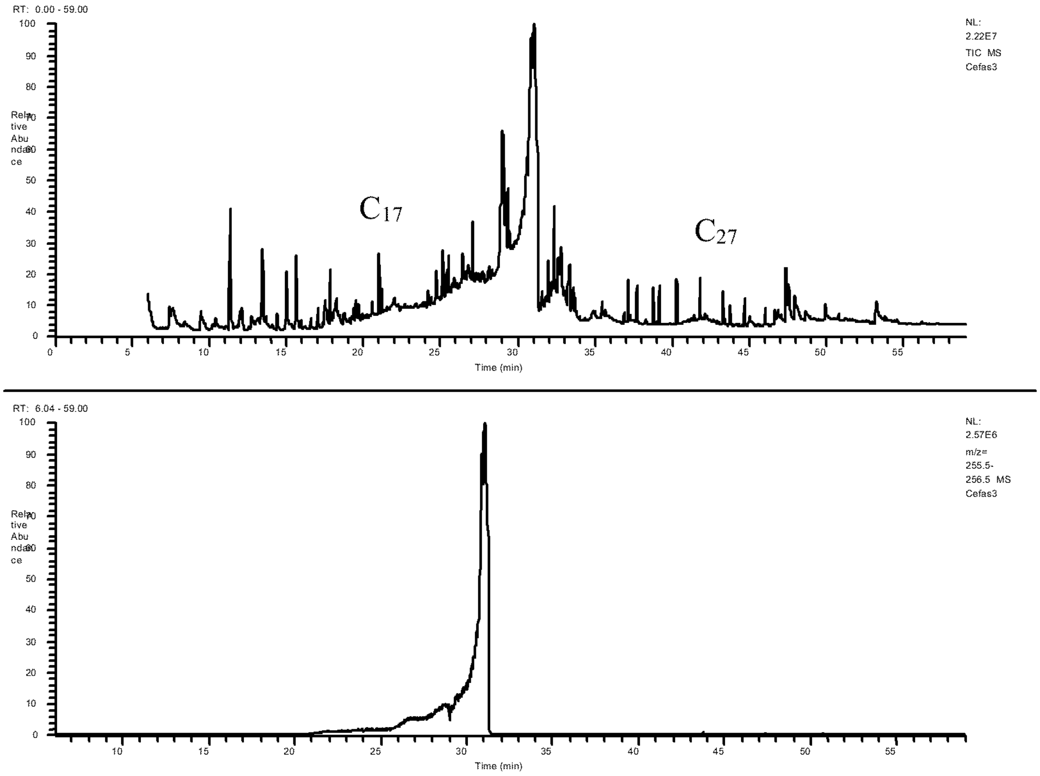


Figure 8.3. GCMS chromatogram for the Ekofisk 2 sample (plots as Figure 8.1).

The total hydrocarbon content of the Ekofisk were both less than the detection limit of 0.1 mg l^{-1}. The *n*-alkanes were present at a concentration of 212 and 93 μgl^{-1} in the samples from Ekofisk 1 and 2 respectively. Naphthalene and C_1-naphthalenes were measured in Ekofisk sample 1 at a concentration of 67 and 109 μgl^{-1} respectively. For the Ekofisk sample 2, naphthalene was present at a concentration of 86 µg/l and C_1-naphthalenes at 110 μgl^{-1}. None of the other PAHs measured in the Ekofisk 1 sample were detected above the method detection limit of 0.1 μgl^{-1}. In the Ekofisk 2 sample by contrast phenanthrene and anthracene were also present at concentrations of 40 and 8.7 μgl^{-1} respectively. The chromatographic peaks following anthracene were difficult to quantify as they were masked by the dominant sulphur peak. To reliably confirm the presence of other aromatic hydrocarbons further sample 'clean-up' would be required.

Of the five metals analysed in the Ekofisk 1 and 2 samples manganese was detected at the highest concentration 178 and 24 μgl^{-1} respectively. All of the metals were present at low concentrations in both samples. The concentration of alkylphenolic chemicals in both samples was also low relative to marine and coastal sites.

All three samples are broadly similar in terms of the range of hydrocarbons measured by fluorescence spectrometry. Because not all oils fluoresce to the same degree, however, there may be some discrimination if the composition of the hydrocarbon content varies across samples. This is a possibility, as the history of the development of drilling fluids in the North Sea shows (see section 8.5 below). Analysis using GC-MS indicated some differences between the three samples which could not be identified further within the scope of the current project. For future studies a quantification of the total hydrocarbons present based on GC-MS would be preferable, but more information would be needed on the drilling history of individual fields if the contributions from different development phases were to be allocated.

The Ekofisk sample 2 shows measurable concentrations of both phenanthrene and anthracene while these chemicals were below the detection limit in both of the other samples.

The trace metal concentrations are shown in *Table 8.3* together with the Environmental Quality Standard level (DoE reports 2986/1, 2685/1, 2686/1 and DETR 4497[1]) set for each of these metals, based on information relating to their toxic effects. It is evident from a comparison of these concentrations with the EQS values set for the protection of aquatic organisms that the trace metal concentrations measured in all of the cuttings water samples fall below the respective EQS values indicating that toxicity due to these elements is unlikely.

The concentrations of alkylphenolic chemicals in each cuttings water sample were similar to the levels of these compounds detected in many surface waters in both the UK and the USA (Wilkinson et. al., 1997[2]).

8.5 INTERPRETATION OF RESULTS

Historically, the early exploration and production wells drilled in the North Sea were drilled using water-based muds. When full development began and wells were deviated in order to enhance oil production within a formation, diesel fuel was added to these water-based muds on the platform in order to aid lubrication of the drill string. This led, however, to widening area of biological impact around North Sea oilfields, and diesel-based drilling fluids were banned as a consequence. Next in line were low-toxicity (alternative) base-oils, mainly formulated using de-aromatised kerosenes. These in turn were banned and succeeded by "synthetic" base-oils based, for example, on esters, ethers, and α-olefins. Fields which have been developed over a long time period (such as the Ekofisk complex) can exhibit residual evidence of historic drilling phases during which all of these types of drilling fluids were used.

The concentrations of the metals measured in the samples are less than the established environmental quality standard (EQS) values, which are based on both the short-term (acute) and long-term (chronic) toxic effects of the individual metals to a variety of freshwater and marine species. There is no EQS value established for manganese in saltwater. However, manganese has been shown to be of low toxicity to a range of marine species (DETR 4497, *op cit.*) and it is likely to be rapidly precipitated and therefore less bioavailable. The concentrations of metals in the cuttings water are likely to be low due to the presence of high concentrations of sulphide in the cuttings piles, since sulphides bind metals and reduce their concentrations in interstitial water. Trace metals that are dissolved into the pore water will also adsorb onto Mn and Fe oxyhydroxides at the sediment water interface.

The nonylphenol concentration was less than the established EQS value (1 $\mu g l^{-1}$) derived for this compound in saltwater (Wilkinson et al., *op cit*). Nonylphenol is acutely lethal to the crustacean *Tisbe battaglai* at levels of 125 $\mu g l^{-1}$ or higher and no effects upon its normal life cycle have been found at a concentration of 31 $\mu g l^{-1}$ nonylphenol (Bechmann, 1999[3]). However effects upon the reproduction of the rainbow trout (*Oncorhynchus mykiss*) have been shown to occur at concentrations of 10 $\mu g l^{-1}$ nonylphenol and 3 $\mu g l^{-1}$ octylphenol (Jobling et al., 1996[4]). The concentration of alkylphenolic chemicals in the cuttings water were therefore around ten times lower than the levels at which effects have been shown to occur.

Neither the trace metals nor the nonyl/octylphenols were therefore present at sufficient concentrations to have produced the observed mortality of *Tisbe* in the Ekofisk 1 and 2 cuttings water samples.

The n-alkanes were present at a lower concentration in the Ekofisk 2 sample which was the sample producing the highest toxicity. However the two polycyclic aromatic hydrocarbons phenanthrene and anthracene were only detected in the Ekofisk 2 sample indicating a possible association with the toxicity of this sample. However there are very few comparative standards established for hydrocarbons against which

[1] DETR 4497/1, 1998. *Proposed Environmental Quality Standards for Manganese in Water.*
[2] Wilkinson, M., Fawell, J.K., Whitehouse, P. and Sutton, A., 1997. *Proposed Environmental Quality Standards for Nonylphenol in water.* R&D Technical Report P42. Foundation For Water Research.
[3] Bechmann, R., 1999. *Effect of the endocrine disrupter nonylphenol on the marine copepod Tisbe battaglai.* The Science of the Total Environment 233. Pp 33-46.
[4] Jobling, S., Sheahan, D.A., Osborne, J.A., Matthiessen, P., Sumpter, J.P.. 1996. *Inhibition of testicular growth in trout exposed to environmental estrogens.* Environ.Toxicol.Chem. 15:194-202.

the concentrations in the cuttings water samples can be compared. There is, however, a UK environmental quality standard (EQS) for naphthalene, a two-ring parent PAH compound. In seawater, the maximum acceptable concentration (MAC) for naphthalene is 80 μgl^{-1}, and this concentration is very similar to the values that we observed in the cuttings water samples (67 - 86 μgl^{-1}). As an annual average the EQS value for naphthalene is 5 μgl^{-1}, which would be exceeded by more than a factor of 10 times if the concentration in the cuttings water were maintained, with a consequent likelihood of biological impacts. No EQS values have been established in the UK for other hydrocarbons or PAH.

The low level of mortality of Tisbe in Ekofisk sample 1, cannot be explained by the concentration of any of the compounds measured. However, the main difference between the Ekofisk 2 cuttings water sample is the presence of anthracene and phenanthrene at concentrations of 8.7 and 40 μgl^{-1} respectively. The cuttings water from sample 2 from Ekofisk produced 50% mortality of the crustacean *Tisbe battaglai* after 48 hours exposure to a dilution of approximately 1 in 3, which would contain approximately 3μgl^{-1} anthracene and 13 μgl^{-1} of phenanthrene. Toxicity studies conducted in the laboratory have shown that another small crustacean species (the brine shrimp, *Artemia salina*) exhibited 50% mortality after 48 hours exposure to a solution containing 32 μgl^{-1} of anthracene or 680 μgl^{-1} of phenanthrene (US EPA, 1989). The measured concentrations of anthracene and phenanthrene therefore only represent 9 and 2 % of these 50% effect concentrations in a 48 hour study. This suggests that these compounds only make a partial contribution to the toxicity of the sample. However, it is possible that there are other PAHs present in the sample that are contributing to the toxicity but that these could not be resolved without further sample clean-up. In any case the full toxicity of the sample cannot be fully explained by the concentrations of those compounds detected in the Ekofisk 2 sample.

Based on chronic (long term toxicity) studies the concentration of anthracene considered to have a negligible effect (NC) was calculated to be 0.0007 μgl^{-1} (Kalf, 1995[1]). In order to reduce the concentration of anthracene in the cuttings water below this NC limit a dilution of 1:12,400 is required.

8.6 RELEASE RATES AND RISK ASSESSMENT

Potential release rates were examined in *Section 7.3.3* where it was concluded that a properly-designed sand layer would be able to accommodate, with a large factor of safety, all of the leachate expelled from the cuttings during the initial, most rapid phase of consolidation (ie. during placement of the sand layer). It was also concluded that, for the majority of piles, the total volume of leachate expelled during long term consolidation could be accommodated by a 2-metre sand layer except, possibly, in the case of some very high piles. The cover design could be modified, if necessary, for such piles.

Rates of release through the completed covers due to the 'pumping' mechanism driven by waves have been estimated to be negligible. However, potentially significant releases have been identified during construction. These arise from the likely disturbance of the uppermost layer of cuttings during placement of the sand layer and are tentatively estimated to be about 4.2 litres/sec.

The released leachate would immediately be diluted by the turbulence induced by sand placement, ie. the same mechanism that gives rise to the disturbance of the cuttings and the leachate release. An estimate of the initial dilution can be derived by assuming that the sand placement progresses over the pile at a speed of 0.17 m/sec. 0.17 m is the diameter of the nominal area of coverage (0.023 m^2) per second at the likely maximum rate of placement. If it is assumed that the leachate is almost immediately mixed into the water over a distance of 0.5m on either side of the line along which sand is being placed and up to 0.5m above the surface of the cuttings, the initial mixing volume would be:

$$(0.5 + 0.5 + 0.17) * 0.17 * 0.5 \text{ m}^3 = 0.099 \text{ m}^3 = 99 \text{ litres}$$

[1] Kalf D.F., Crommentuijn G.H,. Posthumus R., and van de Plassche E.J., 1995. *Integrated environmental quality objectives for polycylic aromatic hydrocarbons (PAHs).* RRIVM Report no.679101018, Bilthoven.

On this basis, the initial dilution would be 99/4.2, approximately 24 times, but it is expected that the initial mixing volume will, in fact, be somewhat greater than 99 litres. The laboratory experiments have shown that, for the most toxic sample of leachate, there was no measurable toxicity over 24 hours at a dilution of 1:20. The initial dilution thus immediately achieves concentrations at which no acute toxic effects were observed and it is concluded that acute toxic effects on marine organisms are unlikely to occur. In fact, concentrations will be lower as the contaminated water will further dilute as it passes away from the area being covered. In addition, exposure times are likely to be of very short duration, even within the footprint of the pile, because sand placement will move around the pile and the current direction will vary during the tidal cycle.

Assessment of the potential for longer term (chronic) effects ideally requires dispersion modelling which is beyond the scope of this component of the JIP. However, a simplistic approach (based on Fischer *et al.*, 1979[1] and Zimmerman,1986[2]) is sufficient to show that the potential for chronic effects is low. The one-dimensional solution for lateral diffusion (parallel to the sea bed) of a unit mass of contaminant with diffusion coefficient K_y released at time $t = 0$, and at $y = 0$ is:

$$C\ (y) = \frac{1}{\sqrt{4\pi t K_y}} Exp(-y^2 / 4K_y t)$$

The one-dimensional solution for vertical diffusion of a unit mass of contaminant released at t=0, z=0 with diffusion coefficient K_z and with an impervious sea bed barrier at $z = 0$ is:

$$C\ (z) = \frac{2}{\sqrt{4\pi t K_z}} Exp(-z^2 / 4K_z t)$$

where C is in units mass/length. The factor 2 arises because the material can only diffuse in one direction because of the sea bed. For short time scales and relatively deep water, the capping effect of the sea surface can be neglected. The two-dimensional solution for diffusion of instantaneous release of mass M at $t = 0$, $y = z = 0$, with diffusion coefficients and sea bed boundary as above, is obtained by multiplying above solutions and normalising so that $\iint C\, dz\, dy = M$ giving:

$$C\ (y,z) = \frac{2M}{4\pi t \sqrt{K_y K_z}} Exp(-y^2 / 4K_y t - z^2 / 4K_z t)$$

In three dimensions, with advection, the two-dimensional solution can be used to obtain the full solution by imagining a thin two-dimensional slab of water moving past the source at $t = 0$ and advecting downstream along the x axis with velocity u. If the input rate is R (mass/second) then neglecting streamwise diffusion (requiring $t >> 2\ K/u^2$), the solution above can be used to obtain:

$$C\ (x,y,z) = \frac{2R}{4\pi x \sqrt{K_y K_z}} Exp[-y^2 u/(4K_y x) - z^2 u/(4K_z x)]$$

where C is in units mass/volume. If R is given as a volume rate, rather than a mass rate, then C is in units (volume contaminant)/(unit volume). The peak concentration (at the seabed $z = 0$ and along the centre line $y = 0$) which is used to determine 'x' once the required C_{max} is given, is then:

$$C_{max}(x) = \frac{2R}{4\pi x \sqrt{K_y K_z}}$$

[1] Fischer H.B., List J.E., Koh R.C.Y., Imberger J., Brooks N.H. (1979), *Mixing in Inland and Coastal Waters.* Academic Press, New York.

2 Zimmerman J.F.T. (1986) *The tidal whirlpool: A review of horizontal dispersion by tidal and residual currents.* Netherlands J. Sea Res. 20 (2/3), 133-154.

From *Section 8.5*, the required dilution is 12,400, i.e. (Volume contaminated water)/(Unit Volume) =1/12 400 = 8×10^{-5} and the estimated release rate is R=4.2×10^{-3} m^3/s. Assuming an average current speed of 0.1 m/s and a water depth of 150m, the required dilution along the centre line of the 'plume' is achieved approximately 90 metres down current of the release point, and about 15 minutes after release. It is therefore concluded that sufficient dilution will take place to ensure that the anthracene concentration is well below the Dutch chronic value (Kalf, op cit).

The tests described here were undertaken on samples of cuttings from just two piles. The concentration of PAHs, metals and other chemicals present in the interstitial water will vary between different cuttings piles and even within individual piles. In order to take some account of variations of interstitial water chemistry, average sediment concentrations have been used to derive equivalent interstitial water concentrations.

Average sediment concentrations for a range of chemicals in ten cuttings piles have been summarised (CORDAH, 1999[1]). From these data the highest measured average concentration of PAHs was 0.128%. A concentration of 0.128% PAH is equivalent to a concentration of 1.28g/kg PAH. The concentration and types of PAH present will be influenced by the compounds used in the original drilling muds and the partitioning and degradation of these compounds following disposal. For simplicity it will be assumed that anthracene is the only PAH present at a concentration of 1.28 g/kg.

The organic carbon content of sediment has been shown to be an important influence on bioavailability of sediment contaminants because most hydrophobic organic contaminants will become adsorbed to the surface of organic matter in the sediment (DeWitt et al., 1992[2], Seth et al., 1999[3]). Calculation of the partitioning of contaminants between the organic carbon and the interstitial water in sediment therefore allows an estimate of the concentration of contaminants that may be released in drill cuttings leachate. The partitioning of anthracene in the cuttings pile can be calculated based on the following equation:

$$PAH_{iw} = PAH_b / K_{oc} \times F_{oc} \quad \text{(Swartz et al., 1995[4])}$$

Where:

PAH_{iw} = the PAH concentration in the interstitial water
PAH_b = the PAH concentration in the bulk sediment (kg dry weight)
K_{oc} = the organic carbon partition coefficient (L/kg)
F_{oc} = the fraction of organic carbon in a sediment (kg dry weight)

The K_{oc} expresses the relative partitioning of a given chemical between sediment organic carbon and water.

For example:

PAH_b =1.28 g/kg
K_{oc} for anthracene = 23442.288
F_{oc} =0.03 kg[5]

$$PAH_{iw} = 1.28 / 23442.288 \times 0.03 = 0.00182\ gl^{-1}\ (1820\ \mu gl^{-1})$$

[1] Cordah, 1999. D*etermination of the Physical Characteristics of Cuttings Piles, using Existing Survey Data and Drilling Information*. Report for the UKOOA Drill Cuttings Joint Industry Project, Cordah Limited, Aberdeen, November 1999.
[2] DeWitt T.H., Ozretich R.J. Swartz R.C. Lamberson J.O. Schults D.W., Ditsworth G.R. Jones J.K.P. Hoselton L. and Smith L.M., 1992. *The influence of organic matter quality on the toxicity and partitioning of sediment associated fluoranthene*. Environmental Toxicology and Chemistry 11, 197-208.
[3] Seth R, Mackay, D. and Muncke J., 1999. *Estimating the Organic Carbon Partition Coefficient and Its Variability for Hydrophobic Chemicals*. Environ. Sci. Technol. 1999, 33, 2390-2394.
[4] Swartz R.C. Schults D.W., Ozretich R.J. Lamberson J.O. Cole F.A. DeWitt T.H., Redmond M.S. and Ferraro S.P., 1995. PAH: *A model to predict the toxicity of polynuclear aromatic hydrocarbon mixtures in field-collected sediments*. Environmental Toxicology and Chemistry 14, 1977-1987.
[5] Northern North Sea sediment organic carbon content falls in a range from 3-10%. The organic carbon content in cuttings piles is likely to be above 10% due to the hydrocarbon content. The value of 3% used here will result in a higher interstitial water concentration and hence a worst case estimate.

In this example it is assumed that anthracene is the only PAH present but, in fact, there are likely to be a range of PAHs present in the interstitial water with the more soluble and, in most cases, less toxic compounds such as naphthalene present in higher concentrations. However based on the presence of anthracene alone at a calculated concentration of 1820 μgl^{-1} in the interstitial water, a dilution of 2,600,000 is required to reduce this concentration below the Dutch chronic value calculated for anthracene.

The simple diffusion algorithm applied earlier suggests that this dilution would be achieved after about 50 hours and 18 km downstream of the cuttings pile (assuming a unidirectional current). Although this estimate is based on conservative assumptions that may be unlikely to occur in practice, and a simplistic description of the diluting mechanisms, it does suggest a potential problem that should be the subject of further study.

8.7 CONCLUSIONS

Only one of the three cuttings water samples produced toxic effects when tested on a representative marine organism. It is considered likely that the highest rates of leachate release will occur during construction of the cover. Application of simple dilution and diffusion algorithms has shown that the immediate dilution of the released leachate, by the disturbing processes that give rise to the release, will be sufficient to avoid acute toxic effects and that sufficient dilution will occur within about 100 metres to avoid chronic toxic effects.

This assessment has been undertaken on a very limited database, restricted to samples from just two piles. A conservative consideration of the potential PAH concentrations in leachate from other piles (assuming that anthracene is the only PAH present) suggests that, in some cases, releases during construction might take a significantly longer time to dilute to an acceptable degree. A greater knowledge of leachate characteristics, and the extent to which they vary from one pile to another, is required. It is recommended that, if additional data obtained in the future tends shows that potentially adverse effects may occur, the cover design procedure should include investigation of the leachate characteristics of individual piles in combination with numerical modelling of dispersion and diffusion.

9 ECOLOGICAL IMPLICATIONS OF COVERING

9.1 INTRODUCTION

This section of the report assesses the ecological implications of covering the cuttings piles *in situ* (excluding the potential impacts of leachate escape which were considered in *Section 8*) and appraises the options for habitat enhancement by varying the physical characteristics of the capping matrix. The following topics are addressed:

- the relationship between physical habitat and community diversity in the northern and central North Sea;
- the habitat diversity provided by the cover; and
- the potential for creating a specific habitat to increase the biomass of commercial shellfish and fin fish.

9.2 NORTH SEA ENVIRONMENT

9.2.1 Physical environment

The physical environment of the North Sea will influence the communities that become established on a cover. The central and northern North Sea gradually deepens from 70 to 140 m in a south-north direction. The seabed topography is variable and there is an irregular pattern of depressions and scattered shallow banks with minimum depths offshore varying from 40 m in the south to 100 m in the north.

The majority of cuttings piles have been created in the central and northern North Sea where a variety of substrate types exists. However, sand, gravel and rock substrata similar to those which might be provided by the cover matrix are of limited extent in the central and northern North Sea and are generally confined to a narrow coastal strip extending from northeast England to north of the Shetland Islands. The cover structure will be therefore physically atypical in the areas where most of the piles are located.

9.2.2 Benthos

Faunal assemblages in the North Sea are influenced by factors such as substratum type, ocean currents and their associated plankton, depth, temperature range and circulation patterns.

The major factors underlying the distribution and abundance of the infauna (animals living in sediment) are related to sedimentary characteristics (ie. particle size and carbon levels). The major determinant of the epifaunal community (animals living on top of sediment and encrusting animals) composition is depth, with sediment composition of less significance (Basford et al., 1990[1]). As most benthic species have pelagic larvae, circulation patterns are important in maintaining these benthic regions (Dyer et al., 1983[2]).

Infauna

In general, diversity and abundance of infauna are highest in the 120-140 m depth zone which is characterised by fine sand containing variable amounts of silt. Higher biomasses are recorded where stronger currents predominate and sediments are coarser (east of Shetland and west of the Norwegian Trough). In the silty sediments of the Fladen Ground and smaller depressions, there exists a predominantly sub-surface, deposit-feeding community, eg. sediment-ingesting polychaetes (marine worms), whereas in areas of coarser sediment, east of Shetland, carnivores (mainly motile, predatory polychaetes) predominate. Over the remaining area, surface deposit feeders (eg sedentary polychaetes and amphipods) are dominant.

[1] Basford D. Eleftheriou A. and Raffaelli D, 1990. *The infauna and epifauna of the northern North Sea.* Netherlands Journal of Sea Research Vol. 25: p165-175.
[2] Dyer M.F., Fry W. G., Fry P. D. and Cranmer, G. J. 1983. *Benthic Regions in the North Sea.* Journal of the Marine Biological Association of the UK. Vol 63, 683-693.

Biomass decreases in a northerly direction, both in terms of total biomass and within each taxonomic group However, total abundance (numbers of animals per m^2) tends to increase towards the north (Marine Laboratory Aberdeen, 1996)[(1)].

Epifauna

Depth is a major factor influencing the distribution of epifaunal species (Dyer et al, Op cit.) and it has been shown that there is a clear difference between the fauna either side of a line from the Tyne to Northern Denmark, which lies just south of the 100 m isobath and demarcates the southerly limit of distribution for a number of species (Dyer et al, Op cit.). Secondary to depth, substrate type has an important impact on epifaunal communities, the clearest contrast being between mobile sedimentary habitats and hard substrates.

The echinoderms (urchins and starfish), are particularly conspicuous within the North Sea epifauna. In addition, mobile crustaceans such as the hermit crabs *Pagurus* sp, decapod shrimps ie. *Spirontocaris lilleborgi* and, where sediment is sufficiently stable, sessile species, eg; sponges, bryozoans (sea mats) such as *Flustra flustra*, tunicates (sea squirts) and anthrozoans (anemones) are found (Dyer et al, Op cit. & Basford et al., Op cit.). One epifaunal species of particular note is the rare, cold-water coral *Lophelia pertusa* which usually occurs in depths in excess of 150 m, but occasionally in shallower inshore waters (Manuel, 1988[2]). The exact nature of the epifauna at a specific location will result from a number of factors, with small scale variability superimposed on larger scale changes correlated to gradients in physical and biological factors.

Taking the above into account and considering the atypical nature of the habitats produced by the cover structure in the context of the North Sea, it is difficult to predict accurately the epifaunal communities which are likely to occur as a result of covering. Colonisation of offshore installations may however provide an insight into the nature of the communities anticipated and is therefore considered below.

9.2.3 Marine fouling on oil and gas installations

Seaweeds dominate the where sufficient light is available ie. to between 10 and 20 m. On deeper surfaces a faunal turf is established consisting of calcareous bryozoans (sea mats) and hydroids (colonial animals). On the parts of the structure below about 70 m, the aggregate tubeworm *Filograna implexa* and deep-water barnacle *Balanus hameri* are common (Fortreath et al, 1982[3]).

Soft corals and anemones may be scattered throughout the hydroid cover and are often in the mid-water depth zone from 30 m to 100 m. The plumose anemone *Metridium senile* has a recorded depth range of 5 m to 140 m. In combination with the soft coral *Alcyonium digitatum,* it is responsible for a sizeable proportion of the biomass in the mid-water depth zone. *Lophelia pertusa* has been observed on North Sea installations.

9.3 PREDICTED COLONISATION OF THE COVER STRUCTURE

Colonisation of the cover structure will occur by settling plankton and migration from surrounding installations. The armour stone covers would encourage a hard substrate, depth dependent, epifaunal rich community to become established. Such communities are uncommon in the North Sea and, other than on installations, are confined to the Fair Isle - Shetland Channel (Basford & Eleftheriou, 1988)[4].

1 Marine Laboratory Aberdeen, 1996. *Environmental Monitoring of the Seas Around Scotland.* Scottish Office Agriculture, Environment and Fisheries Department.

2 Manuel, R.L., 1988. *British Anthozoa Coelenterata: Octocorallia & Hexacorallia*. Published for The Linnean Society of London. E.J.Brill/Dr W. Backhuy: London Academic Press.

3 Fortreath, G.N.R. Picken, G.B. Ralph, R. and Williams, J., (1982). *Marine Growth Studies on the North Sea Oil Platform Montrose Alpha.* Marine Ecology Progress Series Vol. 8: 61-68.

4 Basford, D. and Eleftheriou, A., 1988. *The Benthic Environment of the North Sea.* Journal of the Marine Biological Association UK Vol 68 p125-141.

In addition, sessile invertebrates, echinoderm species such as *Astropecten irregularis* and *Asterias rubens* (starfish), decapod crustacea, molluscs and polychaetes would be expected with varying frequency dependent upon geographical location, depth and substrate. Infauna will colonise the gravel layer, which, due to its physical stability may support a moderately diverse community of filter feeders and mobile predators. Over time the deposition of finer sediment over the gravel will increase diversity of the community present, although this finer material may be subject to periods of winnowing during storm conditions.

9.4 APPRAISAL OF COVER STRUCTURE AS A RESOURCE

9.4.1 Introduction

The creation of the cover structure will effectively create an artificial reef. Artificial reefs mimic the biological and or physical functions of a natural reef and can be designed to enhance marine habitats by, for example, providing suitable feeding, refuge, spawning and nursery areas. In addition, the concentration of commercial species which are often attracted to such reefs can reduce fishing effort.

There is evidence to suggest that oil and gas production platforms, and associated structures, can be important to fish stocks over a much larger area than the structure itself because they can provide protection for the fish during their most vulnerable, juvenile stages.

Operating oil platforms extend throughout the water column, providing benthic, mid-water and surface habitats. Their presence may change relatively unproductive areas into diverse, dynamic and highly productive ecosystems (Aabel et al., 1996)[1]. Although in some cases the structural support pilings may remain *in situ* truncated above the cover surface, the covered piles will not have the same potential because, in comparison, they will be limited in depth range and lack physical habitat diversity.

9.4.2 Habitat enhancement

The replacement of the original ecosystem with a different ecosystem by placing an artificial reef on a homogenous soft bottom, and thus enhancing spatial heterogeneity, can be described habitat enhancement. Whether or not this is desirable depends on the relative values of the original and replacement communities.

The conservation significance of the relatively small area of unnatural habitat created by the cover is doubtful. Although the area of an uncommon North Sea habitat type would be increased (by a very small percentage), the communities colonising the cover are unlikely to have high intrinsic value unless there occurs colonisation by rare species such as *Lophelia pertusa*.

The presence of the armoured pile may further increase habitat diversity by changing the horizontal and vertical profile of the cover thus creating an environment with greater topographical variability than the surrounding seabed areas. Such a structure may provide more shelter for both fish and invertebrates. Infaunal diversity could be enhanced by increasing the range of particle sizes used in the gravel cover. However, the feasibility of this would depend on the hydrographic constraints of the location in question.

9.4.3 Fishery enhancement opportunities

Fin Fish

The maximum pile dimensions recorded by Cordah (1998[2]) are 26 m in height covering an area of 22,246 m^2. A maximum slope angle of 11° is expected producing a profile which will provide limited lee effect or shelter from prevailing currents for any fish in the vicinity. However, the pile will constitute a change in

---

[1] Aabel, J. P., Cripps, S. and Kjeilen, G., 1996. *Oil and Gas Production Structures as Artificial Reefs*. Proceedings of the 1st Conference of the European Artificial Reef Research Network, Ancona, Italy, 26-30 March 1996.

[1] Cordah (1998). *Review of Drill Cuttings Piles in the North Sea. Report for the Decommissioning Communications Project,* Cordah Limited, Aberdeen, November 1998.

seabed topography which may attract demersal (bottom feeding) fish and could be used as a navigational reference, for shelter or for feeding. There is evidence that fish which feed in the vicinity of oil contaminated sediment, may become contaminated and therefore there is potential for tainting to occur (Davies and Kingston, 1992), although this is considered unlikely given the large dilution factor of any leachate, and assuming that contaminated solids are contained by the cap (*Pers comm*. MLA).

Covered areas of seabed are not predicted to be used as spawning or nursery grounds due to the generally specific locations used by spawning fish and the small extent of the covered area.

Shell Fish

Artificial reefs have been used commercially to grow lobsters and edible crabs in coastal areas especially in Japan and the USA and spaces between the capping armour stone could provide habitat for adult crustacea. The deep water location of most piles in the North Sea UKCS is however unsuitable for commercial crustacean species although shallower more coastal areas may be suitable.

9.5 CONCLUSIONS

The conclusions drawn from the study are:

- whilst the cover may provide limited habitat enhancement, it is considered unlikely that fisheries resources could be significantly enhanced;
- the most diverse communities are likely to occur in areas of less than 50 m water depth where only three cuttings piles are located; where conditions allow the capping materials should include a range of particle sizes;
- the ecological significance of cover communities is not predicted to be very high unless colonisation by rare species such as *Lophelia pertusa* occurs;
- given the large dilution factor of any escaping leachate, and provided the cap effectively isolates the contaminated solids, fish taint from the covered pile is considered to be unlikely.

10 MONITORING AND MAINTENANCE OF PILE COVERS

10.1 INTRODUCTION

In Section 6, it was concluded that although the cover could be constructed to withstand, in the short term, impacts of trawling and anchoring and most impacts from objects falling from structures while are only partly removed, there is a risk that long-term cumulative impacts will result in exposure of the cuttings. In addition, structures which are not entirely removed will collapse, although perhaps not for several hundred years, and it is not practical to protect the cover against such impacts. It will therefore be necessary to monitor the covered pile and, if necessary, undertake repairs. The duration of the monitoring programme will depend on the extent, if any, to which the toxicity of the cuttings reduces over time but the possibility exists that the period may be indefinite.

10.2 MONITORING

10.2.1 Objectives and requirements

The objective of the monitoring will be to determine whether or not the armour layer has been damaged and the underlying gravel and sand has been exposed. The armour layer will be about 1 metre thick and the method of surveying thus requires a vertical accuracy of better than ±0.5m. As damage may be restricted to small areas of the cover (eg. a narrow scour caused by a dragging anchor or anchor chain), a high degree of horizontal resolution will be required.

A significant complication which needs to be considered when planning the monitoring operation is the consolidation of the pile due to self weight and the weight of the cover and the consolidation of the underlying seabed materials. These will give rise to apparent reductions of cover elevation of between almost nil and more than two metres (refer to *Figure 7.1*) depending on the original condition of the pile, the thickness of cuttings and the thickness of the cover, all of which will vary over the area of the covered pile. In addition, it must be recognised that there will inevitably be a degree of uncertainty in the estimated rate of consolidation. It will therefore be extremely difficult to determine with confidence how much of the change of cover elevation at any one point is due to consolidation and how much may be due to damage of the cover.

10.2.2 Methods

It is clear that any monitoring based solely on the measurement of cover levels will need to be extremely detailed. It would require the use of ROV-deployed swath systems in conjunction with an array of acoustic transponders located on the seabed around the pile. The acoustic array would need to be related to a permanent datum located to one side of the pile which, in turn, would require monitoring to identify any long term changes of elevation against an absolute datum. The permanent datum may also be subject to damage from, for example, anchors and it would therefore be necessary to establish two or more to provide adequate system redundancy.

Using existing technology based on such a system, it might be possible to survey the cover with an absolute vertical and horizontal accuracy of about ± 0.25m. Repeat surveys may therefore differ in places by as much as 0.5m simply due to inherent inaccuracies. The uncertainty in the estimate of consolidation with time is likely to be at least ±25% of the predicted consolidation even in the case of a cover of uniform thickness over a cuttings pile of uniform thickness and properties. It is therefore not reasonable to expect survey methods based only on level measurements to reveal damage of an armour layer which is only one metre thick. However, damage will in many cases be localised and may be visible on side scan sonar records as an anomalous feature, eg. a long narrow groove resulting from a dragging anchor. Such features may be identifiable using sonar without absolute level control, especially if combined with acoustic seabed classification systems.

There are two possible solutions to this problem. The first is to increase the thickness of the armour stone layer to the point at which it exceeds the error margins inherent in the survey techniques and the estimates of consolidation. In the case of piles which exhibit marginal stability, this may mean that the thickness will need to be increased in stages over a period of several years following construction of the initial cover in order to take advantage of the gradual increase of cuttings strength which will occur as they consolidate.

The second solution is to incorporate within the cover a 'telltale' layer which can easily be detected (eg. by video inspection or, possibly, acoustic methods) if it is exposed. The layer would need to comprise a material with a lifespan which exceeds the period which the cuttings require isolation. This is likely to preclude the use of geotextiles and geogrids which have a limited lifespan. The alternative use of natural materials which contrast with the external armour layer may not be practical. This approach would rely on the visual or acoustic contrast between small armour stones and gravel or sand. In practice the surface texture of a damaged area is likely to be complex with armour stones scattered across the underlayers and may not be easy to detect. In addition, it has been concluded in *Section 6* that a finishing layer of small stone (<75mm) may be desirable in order to minimise the risk of snagging of fishing nets and removal of armour stones.

On balance, it is considered that the best approach is to progressively increase the thickness of armour, following initial construction, to the point at which localised damage which threatens to expose the cuttings will be very obvious as a marked change in the pile morphology between successive surveys. The additional thickness of armour which may be required to achieve this objective might be of the order of 2-3 metres.

10.2.3 Frequency

In principle, the required frequency of monitoring is a function of the:

- estimated frequency of events likely to cause damage;
- the severity of the impacts arising from exposure of the cuttings.

It is recognised that severe damage due to anchoring is likely to occur very infrequently. Damage due to trawling may be more likely and frequent but individual trawling events are likely to cause less damage than anchoring and the main concern is cumulative damage due to repeated incidents. However, the actual frequency of both anchoring and trawling is extremely difficult, if not impossible, to predict. Damage due to major structural collapse is inevitable in those cases where a significant part of the substructure is left in place but, again, the timing is impossible to predict. The required frequency of monitoring must therefore be based on an assessment of the severity of the impacts arising from exposure of the cuttings. For example, if it is concluded that exposure for more than a year is unacceptable, then annual monitoring would be necessary.

10.3 MAINTENANCE

In the event that the cover is damaged, it will be necessary to repair it. As damage is likely to occur some considerable time after construction, it is unlikely that it will be necessary to reconstruct the three-layer cover with a high degree of precision because leachate release rates will be far below those postulated in *Sections 7* and *8*. However, it will be desirable to minimise loss of sediment during repairs and, if the cuttings have been exposed, an initial layer of sand should be placed in the damaged area before the armour layer is reconstructed.

10.4 CONCLUSIONS

Physical monitoring of the covered piles will be required in order to identify and rectify the damage to the armour layer which is deemed to be inevitable in the long term. The limitations of (existing) survey methods and the uncertainty inherent in the estimates of rates of consolidation are such that monitoring of cover elevation is unlikely to be effective if the armour layer is only one metre thick.

On this basis, and recognising that frequent detailed level surveys will be extremely expensive, it is recommended that the thickness of the armour layer be increased to the point at which any damage which threatens to expose the cuttings will be visible as an obvious morphological change between successive surveys without the requirement for a high degree of accuracy in measurements of elevation. In some cases, this additional thickness of armour may need to be applied in stages after initial cover construction in order to prevent instability.

The advantage of this approach is that the monitoring intervals might be longer than would otherwise be the case because the armour layer is less likely to be seriously damaged by individual high impact events and because successive low impact events will take longer to produce significant cumulative damage. The cost of placing the additional armour is likely to be less than the cost of frequent monitoring over an extended period of time.

The thickness of additional armour will vary from pile to pile and depend mainly on:

- the thickness of the cuttings which will approximately determine the uncertainty in the estimated rates of consolidation, and
- whether the structure has been fully or partially removed; partial removal is more likely to give rise to high impact damage events.

An indication of the required thickness can be gained by assuming a 30% uncertainty in the estimate of consolidation rates. The indicative settlement curves presented in *Figure 7.1* suggest that the minimum practical placement thickness (ie. 1 metre) might be added to cuttings piles with a thickness of up to 14 metres. If an additional allowance is made for the error margin in successive comparative surveys (about ±0.5m), it may be concluded that two layers of armour, each of one metre nominal thickness may be required to provide a sufficient margin of safety to ensure that damage to the cover is detected before it becomes serious. However, it is stressed that this figure may vary considerably from one pile to another and, in theory, is also likely to vary over the area of a single pile.

The required frequency of monitoring will depend on the results of an assessment of the magnitude of the impacts of cuttings exposure. The frequency will be great if exposure for short periods is deemed to be unacceptable.

The overall duration of the monitoring programme will depend on the extent to which the cuttings degrade with time (if at all) and become less of a threat to the marine environment.

11 CONSTRUCTION COSTS AND ENERGY BUDGETS

11.1 INTRODUCTION

Although this study has led to the development of cover design requirements, the actual design of a cover, and the volumes of materials which must be placed, will vary from pile to pile depending on the unique geotechnical characteristics and morphology of each pile. Site conditions such as the nature of the underlying seabed, water depths, its location relative to ports where materials can be sourced will also have a bearing on costs and energy consumption. In addition, the prices tendered by contractors may vary depending on the availability of plant and on commercial considerations. Any attempt at estimating construction costs and energy budgets for a limited number of idealised cuttings piles must therefore be viewed with considerable caution.

Cost and energy budget estimates are presented here for the four 'base case' cover designs described in *Section 7.4*. both with and without the additional armouring which may be required in order to facilitate monitoring and minimise the risk of exposure between inspections. The cost estimates are used to identify simple cost trends based on cuttings pile volume and slope angles.

11.2 COST OF CONSTRUCTION

11.2.1 Base case costs

Indicative costs of cover construction have been based on data provided by the three specialist offshore contractors in the Study Team in terms of figures for productivity and operating cost. Costs of raw materials, delivered to barge at the loading station, were also included. As may be expected, the cost of covering will vary considerably depending on the size and shape of the cuttings pile. In particular, the cost of covering expressed in terms of cost per unit volume of cuttings is highly dependent on the size of pile and this cost increases substantially as pile volumes reduce. The cost for the four outline (or 'base case') designs described in *Section 7.4* are summarised in *Table 11.1*.

Table 11.1 Cost estimates for covering four 'base case' piles

Pile type	*Design Volumes*				*Construction cost, £ million*	
	Cuttings	*Sand*	*Gravel*	*Armour*	*Central Sector*	*North Sector*
Large concave	25,500	46,500	27,000	30,000	5.84	6.02
Large conical, steep slope	11,200	18,000	7,000	8,000	2.00	2.05
Large conical, shallow slope	24,500	15,000	20,000	24,500	3.25	3.59
Small, shallow slope	1,500	7,750	10,250	12,500	1.75	1.98

The estimates assume that the structure has been removed sufficiently to permit cover construction using a fall pipe vessel. They are based on year 2001 prices and include for mobilisation, exploitation, standing and loading time. The cost of materials is also included. Due to the range of sizes of plant proposed by the three contributing contractors, the costs have been worked up by producing three separate cost and energy estimates for each site, normalising them for discrepancies in approach and taking the arithmetic average. It has been assumed that construction monitoring surveys will be carried out by the contractors, as the work proceeds, using the spread of plant on site. Two additional surveys, before the works commence and post-construction, have been included.

The cost estimates take account of losses of material during placing. The design volumes shown in *Table 11.1* above have been increased by 20% for sand placement, 15% for gravel placement and 10% for rock placement.

The problem of additional costs arising due to the need to deliver part loads of materials has been addressed by assuming that, in practice, those contractors that have appropriately sized plant will tend to be awarded the work. In addition to this, mixed loads will sometimes be carried by fall pipe vessels. However, it is clear that some cost penalties are likely to be incurred due to size matching problems and the costs have been increased by 15% to account for this.

Construction work is assumed to take place during the summer season, during May to September inclusive, and vessel downtimes have been based on statistics of sea state and exceedence tables provided by Shell (refer to *Section 5*). The difference between construction costs in the North and Central Sectors is due mainly to the different wave climates and the slightly greater delays that can be expected when working in the North Sector.

It is assumed that the cost estimating procedure described above will give figures accurate to +/-15% of the estimated cost. However, this does not show the sensitivity of the costs to changes in other circumstances, such as weather and material losses. To quantify these, two variations have been examined. The first of these is where the weather downtime is 50% lower than expected and the loss of material on placement is also reduced by 50%. The second is where the weather and material losses are 50% greater than assumed. The effect these changes have on the total cost figure is shown below:

Table 11.2 Sensitivity of costs to changing circumstances

Changed Circumstances	***Percentage of Base Cost***	
	Central Sector	***Northern Sector***
Weather and material losses reduced by 50%	90	88
Weather and material losses increased by 50 %	112	115

The effect of the slightly higher weather downtime in the northern sector is evident from these figures. Taken together with the estimated accuracy of the example costs set out in *Table 11.1*, this suggests an overall cost band of about +/- 30%.

11.2.2 Cost trends

The basic cost estimates set out in *Table 11.1* can be used to develop indicative costs for a wide variety of pile volumes and geometries. *Figure 11.1* overleaf shows the approximate cost of covering conical piles with varying volumes and slope angles up to 18^o, which is the assumed maximum design slope for armouring. In this case, it is assumed that all small piles (ie. less than 2,500 m^3) can be treated as conical for the purposes of cost estimation. The cost of covering a pile of any particular volume is clearly heavily dependent on the slope of the pile. Piles with very shallow slopes will cost substantially more per cubic metre of cuttings than those with steep slopes. However, if the slope is greater than 18^o, the costs will increase because additional sand will be required in order to reduce the armour slope angle. The cost of covering piles with markedly concave slopes may be up to twice that of a regular conical pile of similar volume.

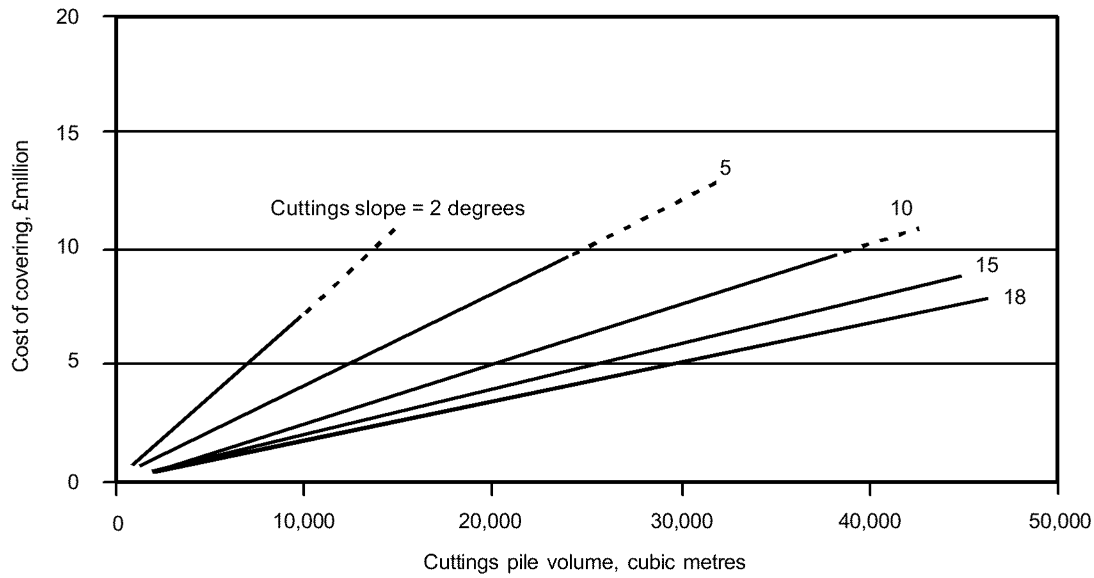


Figure 11.1 Indicative range of covering costs - conical piles.

11.2.3 Cost of additional armour

In *Section 10*, it was concluded that an additional thickness of armour might be desirable in order to reduce the risk of damage to the cover and facilitate the detection of damage during periodic inspections before it reaches the stage where the cuttings become exposed. An additional thickness of 2 metres, possibly placed in stages following the main phase of construction depending on geotechnical constraints, might be appropriate. In the case of conical piles, this additional armour would increase the overall cost of construction by approximately 65%. The cost increase would be less (in percentage terms) in the case of cuttings piles which, prior to initial covering, had concave slopes.

11.3 ENERGY BUDGETS

Energy budgets are presented in terms of Gjoules of power consumed by the construction plant during the covering works, together with an energy equivalent for the materials used (including the material that is wasted due to losses during the construction process). The assumed energy values for the materials are set out in *Table 11.3.*

Table 11.3 Energy values for cover construction materials

Material	*Energy value (GJ) per tonne*
Sand	0.05
Gravel	0.12
Rock armour stone	1.00

Energy usage for the construction plant has been based on figures of estimated energy consumption provided by the contractors. These were provided in the form of kilowatt hours or tonnes of fuel consumed for the different vessels. A conversion check between the two methods of energy assessment indicated that the two gave broadly similar figures for equivalent items of plant. Energy usage has also been broken down into mobilisation, working, loading and stand by. The figures, thus, take account of weather delays.

The energy budgets corresponding to the cost estimates for the four base case piles provided in *Section 11.2* above are set out in *Table 11.4*. For the base case designs with one metre of armour stone, the energy consumption is unlikely to be less than about 50,000 GJ for small piles and may rise to 200,000 GJ or more for the largest piles with volumes of about 35,000 - 45,000m^3. Energy consumption is likely to approximately double if an additional two metres of armour is placed.

Table 11.4 Energy budgets for covering of four base case cuttings piles

Pile Type	Energy Budgets (GJ)			
	Basic design		Including additional armour	
	Central	Northern	Central	Northern
Large pile with concave slope	121,700	122,800	279,600	252,800
Large conical pile with steep slope	39,800	40,000	82,000	82100
Large conical pile with shallow slope	89,300	91,000	218,300	222,200
Small pile with shallow slope	49,100	50,000	114,900	117,100

11.4 CONCLUSIONS

Indicative costs estimates have been prepared which show that the cost of cover construction is strongly dependent on the volume of the pile and its morphology. Small piles with volumes of less than about 2,500 m3, and which generally have shallow slopes, may cost in the region of £1.5M to £2.0M to cover. Large piles with cuttings volumes in the range 30-45,000 m^3 might cost up to £10M depending on slope angles. Costs would increase significantly, by up to about 65% in some cases, if additional armour stone is placed.

Energy consumption is estimated to lie in the range 50,000 GJ to 200,000 GJ or more depending on pile volume and shape if the armour layer is one metre thick. Energy consumption is likely to approximately double if an additional two metres of armour is placed.

12 ASSESSMENT OF POTENTIAL RISKS RESULTING FROM ADOPTION OF IN SITU COVERING

12.1 INTRODUCTION

This section is concerned with the risk to operators, the environment, society, and socio-economic resources from the adoption of in situ covering as a disposal option for drill cuttings piles. The risks are considered in terms of the three main topics: strategic and operational, environmental, and health and safety performance, each of which has been considered in terms of specific issues as follows:

- *Strategic and operational*
  - operational feasibility;
  - established technology / feasibility;
  - security and control of option;
  - established disposal option;
  - public acceptability;
  - compliance with legislation;
  - post-decommissioning maintenance.

- *Environment*
  - containment of contamination;
  - disturbance of sea bed and benthic habitats;
  - pelagic communities and fishing activity;
  - energy use and emissions.

- *Health and safety*
  - personnel safety;
  - navigational/fishing vessel hazard.

The objective of this section is to provide a summary of the risk issues associated with *in situ* covering and to facilitate comparison with other disposal options considered during the JIP studies.

12.2 STRATEGIC AND OPERATIONAL ISSUES

The strategic and operational risks encountered during construction and operation of the *in situ* piles will affect the operators and contractors involved directly in the disposal operations.

12.2.1 Operational feasibility and practicality

The sequence of events during construction would not be complex or involve the use of new and unproven equipment. However, practical and logistical difficulties may be posed by variable pile topography.

Placement of cover materials within the footprint of the installation prior to decommissioning combined with the variation in pile morphologies, from almost perfect cones to markedly concave cones, may cause practical difficulties. The heterogeneity of the piles will result in a requirement for specific designs to be developed in each case based on the pile survey.

Diverse pile morphologies, governed by many factors such as composition, combined with cuttings deposition over soft or loose seabed sediments will have an important influence on behaviour, especially stability. The design of a permanent and stable cover will require the nature of both the pile structure and underlying strata to be investigated in detail.

As a result of geotechnical variance (and uncertainty), the tops of some piles may need to be removed prior to covering in order to render the topography more suitable. In addition, during the decommissioning process, access to seabed level around the installation footings is required for their removal and, in some cases, this will require a considerable proportion of the pile to be relocated.

12.2.2 Established technology

The required technology has been in use for many years in UKCS waters in connection with pipeline burial, free span correction and scour protection but has not been specifically applied to covering cuttings piles during decommissioning projects.

12.2.3 Security and control of construction and operation

The operation will require the co-operation of external organisations including contractors and regulators. As *in situ* capping is installation-specific, external contractors will be under the direct control of the installation operator. Interaction with the regulators (*eg* DTI, CEFAS and HSE) will require levels of collaboration which are comparable with similar offshore operations.

Once the construction has been completed, control will be greatly reduced and the operator will be dependent on the monitoring and survey programme to determine whether the cap is still working effectively.

12.2.4 Established option

In-situ covering, and Contained Aquatic Disposal (CAD) sites with a sediment cap are used in a number of locations world wide. However, covering is not an established option in the UK and has not been used to cover cuttings piles.

12.2.5 Public acceptability

The general public are unlikely to favour a marine isolation treatment and the option may provoke some negative reaction from diverse stakeholder groups such as fishermen's associations. It is anticipated that the expectations of the fishing industry will be to return the areas of cuttings piles to their original status prior to development.

12.2.6 Compliance with legislation

The original deposition of the material on the sea bed is permitted under an exemption to the *Food and Environment Protection Act 1985, Part II (FEPA)*. A licence under FEPA for Marine Construction (for the deposit of substances or articles in the sea) will be required for the fabrication of a cover. There is no reason to assume that, provided that the cuttings are not disturbed during construction, there would be any reason for a licence not to be granted. If construction of the cover involved significant release of contamination, it is unlikely that granting of a licence would be favoured.

12.2.7 Residual liability

In the event of a loss of containment the capped cuttings piles will represent a potential liability in terms of;

- hazard to fishing vessels;
- damage and contamination of fishing gear;
- contamination of the sea bed and water column, and
- public perception.

No limit can be set on the liability and it could present a significant commercial disadvantage to the operator.

12.2.8 Post-decommissioning monitoring and maintenance

A post-construction monitoring and maintenance programme will need to be established to examine the piles for integrity. After an initial monitoring period, monitoring frequency may be reduced over time, as long as the cover remains intact. It is anticipated that maintenance would be required to repair damage caused by trawling gear, anchoring and, in some cases, structure collapse.

12.3 ENVIRONMENT

Environmental impacts associated with the construction and operation of the *in-situ* covering option are the concern of all stakeholders including the operator, the general public, regulators and commercial fisheries interests. The assessment below is based on a generic cuttings pile design and assumes an offshore location.

12.3.1 Containment of contamination

Some of the cuttings material is likely to be liberated during construction of the cover. The amount will be dependent on the geotechnical properties of the pile, construction methods and requirement to remove the apex of the pile cone. Once the cover has been constructed, provided it is stable, it is predicted to maintain integrity unless subjected to severe damage from trawling or anchoring. Significant loss of integrity is only expected to result from repeated damage. The results of the leaching studies suggest that in the absence of disturbance, there is a low probability of contaminated leachate escaping the covered pile structure, other than that released during the covering process. It is anticipated whilst the cover remains intact, the containment of the contents will be greatly increased in comparison with un-covered piles. It is predicted that transfer of contaminants through the food chain is unlikely as the cover would effectively isolate the contaminated material.

12.3.2 Seabed disturbance and benthic communities

The area of the cover will be greater than the existing pile and this additional area will cover previously undisturbed sediment. The additional area of sea bed affected will be comparatively small, however impacts due to the disturbance are likely to be effectively permanent.

There will be a temporary impact over an area greater than the cover caused by settlement of fine particulate matter from covering material during the construction process.

The nature of the community which eventually colonises the cover will depend on the nature of the covering material. Once colonisation of the newly deposited cover is complete it is anticipated that the infauna and epifauna will be more diverse than that originally occupying the more contaminated area of the cuttings pile. It is unlikely that the community will have significant conservation significance.

12.3.3 Pelagic communities and fishing activity

It is expected that the pelagic community associated with the offshore installation will disperse on its removal. The cover may have some limited value to fish as a habitat. Change in the use of the area subsequent to decommissioning would depend on whether the installation is removed to seabed level. In any situation where part of the structure remains protruding above pile level, or hollows are created by the irregular shape of certain piles, a hazard will be presented to trawling (specifically beam and otter trawl gear) vessels. Such structures may endanger the vessel and crew trawling the site. Trawling over the rock armouring, whether deliberately or accidentally is likely to damage both the trawling gear and the cover itself. If trawling can be avoided, or excluded from covering areas subsequent to the removal of the offshore installation (requiring a surface navigation buoy to mark the hazard) then there will be no change relative to the current situation. In the event that the cover is breached by trawling then damage and contamination of fishing gear is likely to result.

12.3.4 Energy use

There will be a small rise in energy consumption and resulting emissions from vessel activity during cover construction, monitoring and maintenance.

12.4 HEALTH AND SAFETY

The consideration of health and safety issues relates to the construction, monitoring and maintenance of the covered structures. Public health issues are not considered here as it is deemed that exposure of the public to contaminated cuttings is highly unlikely to occur.

12.4.1 Personnel safety

The risk to personnel will be similar to other offshore activities such as pipe laying and dredging and offshore maintenance. Providing that no diving is undertaken, the increase in risk would relate to the additional time required for these activities.

12.4.2 Navigational/fishing vessel hazard

The retention of structural footings of offshore installations will cause a navigational hazard for fishing vessels as discussed above. In addition, the rock armouring may also damage fishing gear. There will be risk to fishermen from the cover and from any parts of the offshore installation remaining on the sea bed. In the absence of a surface navigation marker it is likely that damage to gear and risk to vessels will arise.

12.5 CONCLUSIONS

While covering of contaminated sediments (both *in situ* sediments and dredged materials placed in confined aquatic disposal facilities) has been undertaken at several locations around the world, there are no known precedents for the containment of drill cuttings by covering. However, the construction of covers is considered to be feasible and will rely upon established techniques. Some difficulties are however anticipated with the practical application of these techniques in some circumstances. Primarily, difficulties are foreseen if the construction is undertaken with the installation in place. In addition, it is predicted that construction over cuttings piles with steep angles of repose or concave slopes may not be possible without prior removal of the cuttings pile top.

Construction will require regulatory permitting. The success of an application is likely to depend on the extent of disturbance of the cuttings during construction. If there is likely to be a significant release of contamination during construction then permission is not anticipated to be granted. Following construction the covered pile will require monitoring to ensure that it has remained intact. If solids containment is lost, then a period of months or years could elapse before this is observed and remedial measures are taken and thus security of containment cannot be ensured. The covered cuttings piles will constitute a long term liability for the operator.

The cover structure would potentially provide an improved benthic habitat in relation to the contaminated sediment of the deposited cuttings although, it is not likely to be of significant ecological importance or commercial value as a fisheries stock resource. Containment of contamination is expected to be efficient unless the cover is damaged by fishing activity. Some accidental trawling of the cutting piles would be anticipated and damage may occur to the cover structure therefore, there is the potential for damage or contamination of fishing gear or danger to fishing vessels. However, covered structures are likely to isolate the solids contamination if undisturbed.

Impacts on energy use, atmospheric quality and health and safety are not predicted to be significant.

13 SUMMARY OF CONCLUSIONS

This detailed review of the *in situ* covering option has concluded that covering of cuttings piles is not likely to be a practical option in the case of concrete gravity structures which are fully removed. The conclusions that are summarised below therefore concern only cuttings piles associated with concrete structures which are partially removed and those under fixed steel structures.

13.1 CONSTRUCTION MATERIALS AND METHODS

13.1.1 Conclusions

Using existing proven methods, the covers can only be constructed using natural granular materials such as sand, gravel and stone.

Where the installation has been substantially removed, these materials can be placed with the required degree of accuracy using fall pipe vessels. Trailing suction hopper dredgers could also be used to place the initial layer of sand but accuracy of placement would be poor and losses of material would be greater than those arising from the use of fall pipe vessels. Very coarse armour, if required, could be placed using side dump vessels. However, armour coarser than that which can be placed using fall pipe vessels is unlikely to be required.

At present, it is not considered to be practical to construct covers underneath existing installations although it may be possible to place the initial sand layer using ROV systems working from a stockpile of sand placed on the seabed adjacent to the installation. This approach has yet to be proven and is likely to be very slow and costly. In theory, it may also be possible to place a sand layer under an installation using a trailing suction hopper dredger but accuracy of placement is likely to be unacceptably poor (leading to potential cuttings instability) and material losses may be substantial.

13.1.2 Aspects requiring further investigation

If it is shown that a high degree of protection of the cuttings is necessary during decommissioning, it will be necessary for specialist offshore contractors to develop methods of cover construction underneath the structures.

13.2 GEOTECHNICAL ISSUES

13.2.1 Conclusions

The limited amount of reliable geotechnical data which is presently available does not permit the development of any firm, widely applicable conclusions concerning the limitations of the *in situ* covering option. However, it appears possible that the existing stability of some cuttings piles may be marginal.

The data which are available, in combination with consideration of cuttings pile morphology and the manner in which the piles were created, suggests that large conical piles may comprise relatively sandy materials. Large piles with markedly concave slopes are likely to comprise predominantly fine materials. It is the latter which tend to exhibit the steepest local slopes and which are thought likely to have the least favourable geotechnical characteristics. Small piles appear generally to be characterised by relatively shallow slopes (which are not necessarily indicative of any particular composition) but often have small pinnacles of material supported by parts of the structure.

At present, it is thought likely that it will be necessary either to 'build out' marked concavities in the pile surfaces or to remove the tops of the piles so that the slope of the completed sand layer is no more than 11^o. The gravel and armour protection layers can then be placed with a uniform thickness over the sand layer. In some cases, building out the concavities may not be sufficient to prevent instability of the cuttings because of the considerable thickness of sand which will need to be placed. Removal of the top of the pile may be the only way in which the load imposed by the sand can be reduced.

In the case of conical piles with relatively uniform slopes (suggestive of better geotechnical conditions), it is assumed that steep slopes will need to be reduced to a maximum of 18^o by varying the thickness of sand prior to placement of the armour layers. This assumption is based on consideration of the methods of armour stone placement and the size of armour required to protect steep slopes

13.2.2 Aspects requiring further investigation

Additional investigation of the geotechnical properties of cuttings piles is required in order to appreciate fully the geotechnical limitations of *in situ* covering. It is likely that each pile for which this solution is considered will need to be investigated. Geotechnical investigations should include the substrate on which the piles rest. It is important that future geotechnical investigations define the *in situ*, undisturbed properties of the cuttings, possibly by means of cone penetration testing if it proves impractical to obtain undisturbed samples, and are undertaken in accordance with recognised geotechnical standards.

13.3 CONTAMINANT LOSS

13.3.1 Conclusions

Covers constructed using natural materials will be effective in preventing loss of cuttings particles and biological access to the cuttings as long as they do not suffer significant damage. The sand layer can be designed to ensure that its capacity is more than adequate to accommodate all of the leachate that will be expelled from the cuttings. However, being permeable, the covers will not be able to prevent, in the long term, some release of the contaminated fluids which they contain.

These releases will arise from the very slow process of chemical migration and by water exchange induced by a 'pumping' mechanism that is generated by waves passing over the covered piles. This latter process will give rise to negligible losses because the reciprocating nature of the process results in rapid dilution of the leachate in the upper parts of the sand layer.

Potentially more significant pore water releases may occur during cover construction due to disturbance of the upper layer of cuttings. An indicative rate of release of 4.2 litres/sec has been derived. This will occur only while the sand layer is being placed.

One of the three cuttings water samples tested during this study produced toxic effects when tested on a representative marine organism. Application of simple dilution and diffusion algorithms has shown that the immediate dilution of the released leachate, by the same disturbing processes that give rise to the release, will be sufficient to avoid acute toxic effects. Further dilution, sufficient to avoid chronic toxic effects, will occur within about 100 metres of the release point.

A conservative consideration of the potential PAH concentrations in leachate from other piles (assuming that anthracene is the only PAH present) suggests that, in some cases, releases during construction might take a significantly longer time to dilute to an acceptable degree.

13.3.2 Aspects requiring further investigation

The apparent acceptability of low rates of contaminated pore fluid release during construction is based on a limited number of chemical analyses and toxicology tests undertaken during this study. It is necessary to expand this database before conclusions can be drawn concerning the number of piles for which this conclusion is valid.

This study has necessarily been undertaken on the assumption that there is a requirement for long term physical isolation of the cuttings and the prevention of direct access to the cuttings. This may not in fact be the case and, particularly, it is possible that over time, the nature of the cuttings and contaminants will change in a manner which reduces the degree of isolation which is required. The degree of isolation which is required is beyond the scope of this component of the Phase II JIP but is clearly a key consideration in the evaluation of the overall viability of *in situ* covering as a solution to the drill cuttings problem.

13.4 COVER PROTECTION

13.4.1 Conclusions

If the sand layer is protected by at least one metre of gravel and one metre of armour stone, it will be able to withstand the impacts of severe storms. It will also be able to withstand occasional trawling events but cumulative damage may occur in the long term which may give rise to a maintenance requirement.

Dropping and dragging of anchors over the armoured pile will give rise to limited damage but cumulative impacts may be severe. Severe damage may be caused by a single emergency anchoring event in which a large ship drops its anchor close to the pile and drags the chain over the cover. However, the risk of occurrence of such an event is considered to be extremely low.

The armour layer is estimated, in theory, to be able to withstand the impacts of collapse of steel structures which are only partly removed. However, vary large items would penetrate most of the cover and repeated impacts may well result in exposure of the cuttings. It is not considered practical to provide effective protection against collapse of concrete structures which are only partly removed.

Consideration of peak particle velocities under storm conditions shows that there is a risk of damage to covers during construction. Storm events may result in partial loss of an unprotected sand layer and, depending on the site characteristics and the severity of the storm, an unprotected gravel layer. As far as is practical, construction should be scheduled during a period in which conditions are forecast to be relatively benign and the sand and gravel layers should be armoured as soon as possible after placement.

It is not possible to guarantee that the cover will remain undamaged in the long term and, in the event that other studies indicate a requirement for long term isolation of the cuttings, it will therefore be necessary to conduct periodic inspection and maintenance.

13.4.2 Aspects requiring further investigation

It is unlikely that the conclusions concerning this aspect of the covering option will change as a result of further study. However, the requirement for long term isolation has yet to be established and it is recognised that, if it is demonstrated to be unnecessary, the implications of the deterioration of the cover integrity (if not monitored and maintained) will become less significant.

13.5 IMPLICATIONS OF DECOMMISSIONING SCENARIOS

13.5.1 Conclusions

Complete removal of installations will almost certainly require the removal or displacement of a proportion of the cuttings in order to provide access to the lower parts of the structure, the template and connections to pipelines and services. In some cases, the amount of material involved may be a significant proportion of the total. If this material is displaced, rather than recovered to the surface for disposal, the geotechnical properties will be very poor and will exacerbate the difficulty of covering.

Because the cover cannot be placed using existing proven technology until most of the structure has been removed, there exists the risk of disturbance of the exposed cuttings during decommissioning due to impacts from falling objects.

Although not specifically within the remit of this study, it is noted that partial decommissioning which results in parts of the substructure protruding above the cover presents a potential risk to fishing vessels unless some method can be devised to ensure that they cannot deliberately or accidentally trawl over the site.

13.5.2 Aspects requiring further investigation

It is unlikely that the conclusions concerning this aspect of the covering option will change as a result of further study.

13.6 MONITORING AND MAINTENANCE

13.6.1 Conclusions

It is not practical to construct a cover which is resistant to damage from occasional severe impacts (eg. emergency anchoring of large vessels) and from cumulative damage by repeated low impact events (eg. repeated trawling). If it is shown that isolation of the cuttings is required in the long term, it will therefore be necessary to implement a programme of periodic inspection and, when required, maintenance.

The required frequency of inspection will be determined by the results of the assessment of potential impacts arising from exposed cuttings. It will be possible to extend the periods between inspections by increasing the thickness of the armour layer beyond the minimum 1 metre thickness which is assumed in the basic designs developed in this report.

13.6.2 Aspects requiring further investigation

As already noted, the monitoring and maintenance burden may be reduced or even eliminated if other studies show that long term isolation is not required.

13.7 CONSTRUCTION COSTS

Construction costs are heavily dependent of pile volume and shape. For covers with a 1-metre armour stone layer, construction costs are likely to lie in the range of £1.5M for small piles to £10M for the largest piles. Costs may be up to 65% higher if an additional thickness of 2 metres of armour stone is placed.

13.8 ENERGY BUDGETS

Energy consumption is estimated to lie in the range 50,000 GJ to 200,000 GJ or more, depending on pile volume and shape, if the armour layer is one metre thick. Energy consumption is likely to approximately double if an additional two metres of armour is placed.

13.9 ECOLOGICAL IMPLICATIONS

Although covered piles may provide limited habitat enhancement, it is considered unlikely that fisheries resources could be significantly enhanced and the ecological significance of cover communities is not predicted to be very high unless colonisation by rare species such as *Lophelia pertusa* occurs.

Given the large dilution factor of any escaping leachate, and provided the cap effectively isolates the contaminated solids, fish taint from the covered pile is considered to be unlikely.

13.10 ASSESSMENT OF RISKS

A review of the risks of covering cuttings piles has concluded that, as long as the cuttings are physically isolated for the period during which they pose a threat to the environment, covering is a generally low risk option which can be achieved using proven methods of construction and with little or no adverse impact on the marine environment.

However, adoption of this option represents a potential liability due to the possibility of damage of the cover resulting in release of contamination and a risk to fishing activities. The general public are unlikely to favour a marine isolation treatment and the option may provoke some negative reaction from diverse stakeholder groups such as fishermen's associations.

Covering would require regulatory permitting which is likely to be granted if it is shown that disturbance of the cuttings during construction would not be significant.

13.11 OVERALL CONCLUSION

The study team conclude that, on the basis of the data that are presently available, *in situ* covering remains a practical option but there exists a large degree of uncertainty concerning geotechnical constraints. Much work is required to enhance our knowledge of the stability of cuttings piles and it must be recognised that, when this knowledge becomes available, it may be found that covering may not be possible for all cuttings piles.

On the assumption that the cuttings require physical isolation and will not degrade with time to the extent that they no longer present a threat to the environment, adoption of the covering option will give rise to a potentially open-ended requirement for monitoring and maintenance. The key issues that must be addressed before any firm conclusion can be drawn concerning the overall viability of this option therefore focus on the degree and duration of isolation that is required. The degree of protection afforded by covers can be summarised as follows:

Steel - full removal

- long term protection against environmental impacts;
- medium term protection against deliberate and accidental trawling impacts;
- low protection against extremely rare emergency anchoring impacts.

Steel - partial removal

- long term protection against environmental impacts;
- medium term protection against accidental trawling impacts (deliberate trawling most unlikely);
- low protection against extremely rare emergency anchoring impacts;
- moderate protection against structural collapse (100-200 years hence?).

Concrete - partial removal

- long term protection against environmental impacts;
- medium term protection against trawling impacts;
- low protection against extremely rare emergency anchoring impacts;
- very low protection against structural collapse (200+ years hence?).

REFERENCES

Aabel, J. P., Cripps, S. and Kjeilen, G., 1996. *Oil and Gas Production Structures as Artificial Reefs*. Proceedings of the 1st Conference of the European Artificial Reef Research Network, Ancona, Italy, 26-30 March 1996.

Alfonso, A., 1996. *Artificial Reefs for Habitat Protection*. Report of the Results of EARRN Workshop 2 : Management of Coastal Resources and Fisheries Enhancement. Proceedings of the 1st Conference of the European Artificial reef Research Network, Ancona, Italy, 26-30 March 1996.

AURIS Environmental, 1992. *Environmental survey of the Beatrice field, block 11/30, 1992.*

Baines, M., Dec 2000. Pers comm. Independent.

Basford, D. and Eleftheriou, A., 1988. The Benthic Environment of the North Sea. Journal of the Marine Biological Association UK Vol 68 p125-141.

Bechmann, R., 1999. *Effect of the endocrine disrupter nonylphenol on the marine copepod Tisbe battaglai.* The Science of the Total Environment 233. Pp 33-46.

BGS, 1998. *Fulmar Alpha Drill Cuttings Mound – Moisture Content and Particle Size Analysis*. Report to Shell Research Ltd. BGS Technical Report WB/97/27C, British Geological Survey Edinburgh.

BMT, 1999. *Study Research Area 3.2: Mathematical Model Of Pile Dispersal following Disturbance.* Unpublished Report for UKOOA Drill Cuttings Initiative.

Brandsma M.G. and McKelvie S., 1994. *Modelling North Sea Oil based mud cuttings discharges to assess environmental* loading: E&P Forum study. Paper OTC 7395 presented at the 1994 Offshore Technology Conference. Proc. Vol. 1. Houston, Texas.pp.215-222.

Brannon, J.M., Hoeppel, R. E., Sturgis, T.C., Smith I., jnr and Gunnison, D., 1985. *Effectiveness of capping in isolating contaminated dredged material from biota and the overlying water.* US Army Corps of Engineers Waterways Experiment Station, Technical Report D-85-10.

British Standards Institution, 1990. *Methods of test for soil for civil engineering purposes (BS 1377:1990 Parts 1-9).* HMSO, London.

British Standards Institution, 1981. *Code of practice for site investigations (BS 5930).* HMSO, London, 88pp.

British Standards Institution, 1991. *British Standard Code of Practice for Maritime Structures BS 6349: Part 7: 1991 - Guide to the design and construction of breakwaters.* HMSO, London.

Britsurvey, 1998. *Drill Cuttings Survey 1997.* Report for Shell U.K. Exploration and Production. Britsurvey, Great Yarmouth, March 1998.

Brown & Root Survey, 1992. *North West Hutton Drill Cutting Survey – Survey Report.* Report by Brown and Root Survey Ltd, Aberdeen, to Amoco (UK) Exploration Co., November 1992.

Cordah, 1998. *Review of Drill Cuttings Piles in the North Sea.* Report for the Offshore Decommissioning Communications Project, Cordah Limited, Aberdeen, November 1998.

Cordah, 1999. *Determination of the Physical Characteristics of Cuttings Piles, using Existing Survey Data and Drilling Information*. Report for the UKOOA Drill Cuttings Joint Industry Project, Cordah Limited, Aberdeen, November 1999.

Csiti, A., 1993. *Land disposal of Contaminated Dredged Material and Related Issues: State of the Art Report*. Terra et Aqua, No. 53, December 1993, IADC, The Hague.

CUR/RWS, 1995. Report 169: *Manual on the Use of Rock in Hydraulic Engineering*. A A Balkema, Rotterdam.

Dames & Moore, 1999. *Historical review of drill cuttings at the Greater Ekofisk field.* Dames & Moore Report for Phillips Petroleum. Dames & Moore: Stavanger, Norway. 16 pp. + apps

Davies J. M. and Kingston P. F., 1992. *Sources of Environmental Disturbance*. In North Sea Oil and Environment. W.J. Cairns Ed., Elsevier Applied Science.

De Jong, S. A., Zevenboom, W, Van Het Groenwoud, H and Daan, R., 1991. *Short-and long-term effects of discharged OBM cuttings with and without previous washing, tested in field and laboratory studies on the Dutch continental shelf, 1985-1990*. SPE paper 23353. In: Proceedings of 1st International Conference on Health, Safety and Environment, Hague 1991.

Delvigne, G. A. L., 1995. *Laboratory investigations on the fate and physicochemical properties of drill cuttings after discharge into the sea.* In: The Physical and Biological Effects of Processed Oily Drill Cuttings. Metocean Report No.2.61/202 to the E&PForum, pp.16-24.

Department of Energy, 1990. *Offshore Installations: Guidance on Design, Construction and Certification*. HMSO, London.

DETR 4497/1, 1998. *Proposed Environmental Quality Standards for Manganese in Water.*

de Vos, K., W. van Crombrugge, and B. Malherbe (1997). *Environmental Impact of the storage of contaminated dredged material - two case studies*. In: International Conference on Contaminated Sediments. September, 1997. Rotterdam, The Netherlands. Pre-prints. pp. 981-991.

DeWitt T.H., Ozretich R.J. Swartz R.C. Lamberson J.O. Schults D.W., Ditsworth G.R. Jones J.K.P. Hoselton L. and Smith L.M., 1992. *The influence of organic matter quality on the toxicity and partitioning of sediment associated fluoranthene*. Environmental Toxicology and Chemistry 11, 197-208

DWQ 9026, 1992. *Revised Environmental Quality Standard for Copper in Water.*

Ekelund R., Granmo A., Magnusson K., Berggren M., and Bergman A., 1993. *Biodegradadtion of 4-nonylphenol in seawater and sediment*. Environ. Pollut 79 59-61.

Environmental Resources Management, 1999. *The Practicality of Covering Drill Cuttings In Situ*. Report to UKOOA under Task 5.1 of Phase 1 of the UKOOA Drill Cuttings JIP

ES 9026, 1993. *Revised Environmental Quality Standards for Zinc in Water.*

ES 9378, ES 9026, 1992. *Revised Environmental Quality Standard for Nickel in Water.*

Escarameia, M, and May, R.W.P., 1992. *Channel Protection: Turbulence Downstream of Structures*. Report SR 313, HR Wallingford, Wallingford.

Fischer H.B., List J.E., Koh R.C.Y., Imberger J., Brooks N.H., 1979. *Mixing in Inland and Coastal Waters*. Academic Press, New York.

Zimmerman J.F.T., 1986. *The tidal whirlpool: A review of horizontal dispersion by tidal and residual currents*. Netherlands J. Sea Res. 20 (2/3), 133-154.

Fortreath, G.N.R. Picken, G.B. Ralph, R. and Williams, J., 1982. *Marine Growth Studies on the North Sea Oil Platform Montrose Alpha*. Marine Ecology Progress Series Vol. 8: 61-68.

Fortreath, G.N.R. Picken, G.B. Ralph, R. and Swain, G., 1983. *Fouling Below 10 Metres on the European Continental Shelf*. Progress in Underwater Technology: Proceeding of the Subsea Challenge Conference, Amsterdam. Paper C14, Society for Underwater Technology, London.

Gardline Surveys, 1999. *Preliminary investigation of the cuttings pile at the Beryl A platform*. For Mobil North Sea Ltd.

Giere, O. 1993. *Meiobenthology - the microscopic fauna in aquatic sediments*. Springer-Verlag, Berlin, 328 pp.

Grampian Soil Surveys (Aberdeen) Ltd., 1993. *Laboratory Testing – Seabed Samples, Mud Mound, Heather Alpha Platform*.

Higgins, R.P.& Thiel, H. (eds.), 1988. *Introduction to the study of meiofauna*. Smithsonian Institution Press, Washington, London 488 pp.

IOE, 1998. *Investigation of the chemical and physical nature of the cuttings pile beneath the Clyde platform*.

Isbash, S V & Khaldre, K Y., 1970. *Hydraulics of River Channel Closure*. Butterworths, London.

Jobling, S., Sheahan, D.A., Osborne, J.A., Matthiessen, P., Sumpter, J.P.. 1996. *Inhibition of testicular growth in trout exposed to environmental estrogens*. Environ.Toxicol.Chem. 15:194-202

Jones B.R. and Laslett R.E., 1994. *Methods for analysis of trace metals in marine and other samples*. Aquatic Environment Protection: Analytical Methods Number 11.

Kalf D.F., Crommentuijn G.H,. Posthumus R., and van de Plassche E.J., 1995. *Integrated environmental quality objectives for polycylic aromatic hydrocarbons (PAHs)*. RRIVM Report no.679101018, Bilthoven.

Kelly C.A., Law R.J., and Emerson H.S., 2000. *Methods of analysing hydrocarbons and polycyclic aromatic hydrocarbons (PAH) in marine samples*. Science Series aquatic Environment Protection: Analytical Methods No.12

Law R.J., and Blackman, R.A.A., (1981). *Hydrocarbons in water and sediments from oil-producing areas of the North Sea*. ICES CM 1981/E: 16. 20pp. (mimieo)

MacIntyre A.D., 1978. *The Benthos of the Western North Sea*. Rapp.P.-v. Réun. Cons. Int. Explor. Mer, 172: 405-417.

Manuel, R.L., 1988. *British Anthozoa Coelenterata: Octocorallia & Hexacorallia*. Published for The Linnean Society of London. E.J.Brill/Dr W. Backhuy: London Academic Press.

Marine Laboratory Aberdeen, 1996. *Environmental Monitoring of the Seas Around Scotland*. Scottish Office Agriculture, Environment and Fisheries Department.

Maynord, S.T., 1993. *Corps Riprap Design Guidance for Channel Protection*. Paper 3 of River, Coastal and Shoreline Protection: Erosion Control using Riprap and Armourstone, International Riprap Workshop, Fort Collins, July 1993, John Wiley, Chichester.

Neilson, A.H., 1998. *PAHs and Related Compounds*. Biology. Springer-Verlag Berlin. ISSN 3-540-63422-3

Pemberton, G., S., Risk, M., J., and Buckley, D.E., 1976. *Supershrimp. Deep bioturbation in the Straits of Canso, Nova Scotia*. Science 192 790-791.

Pers com., Marine Laboratory Aberdeen, Jan 2001.

Pilarczyk, K W., 1990. *Stability Criteria for Revetments*. Proc. National Conference on Hydraulic Engineering, American Society of Civil Engineers.

Rance, P J & Warren, N F., 1968. *The Threshold of Movement of Coarse Materials in Oscillatory Flow*. Proc. 11th Conference on Coastal Engineering, American Society of Civil Engineers.

Roberts, M., 1998. *Lophelia pertusa - a cold water coral*. WWF & Wildlife Trusts Marine Factsheet.

Roe, T.I. and Johnsen, S., 1996. *The Norwegian Oil Industry Association (OLF): Discharges of produced water to the North Sea*. In: M. Reed S. Johnsen (Eds), Produced Water 2: Environmental Issues and Mitigation Technologies. Plenum Press, London, New York, pp. 257-269.

Rogaland Research, 1998. *Disposal of Oil-based Drill Cuttings*. Report for the Norwegian Oil Industry Association, Rogaland Research – Dames & Moore – Cordah, April 1998.

Servos M. R., 1999. *Review of the Aquatic Toxicity, Estrogenic Responses and Bioaccumulation of Alkylphenols and Alkylphenol Polyethoxylates*. Water Qual. Res. J. Canada. Volume 34, No. 1, 123-177.

Seth R, Mackay, D. and Muncke J., 1999. *Estimating the Organic Carbon Partition Coefficient and Its Variability for Hydrophobic Chemicals*. Environ. Sci. Technol. 1999, 33, 2390-2394

Soulsby, R.L., 1997. *Dynamics of Marine Sands: A Manual for Practical Applications*. Report SR 466, HR Wallingford, Wallingford.

Subsea Offshore Ltd., 1996. Mobil Beryl Field, 1996 Sonar Graphics Survey Results. Report No AB-R-RP-00132 to Mobil North Sea, Subsea Offshore Ltd., Aberdeen.

Swartz R.C. Schults D.W., Ozretich R.J. Lamberson J.O. Cole F.A. DeWitt T.H., Redmond M.S. and Ferraro S.P., 1995. PAH: *A model to predict the toxicity of polynuclear aromatic hydrocarbon mixtures in field-collected sediments*. Environmental Toxicology and Chemistry 14, 1977-1987

Trevor Jee Associates, 1996. *Fishing Equipment*. Report number SH01R10A. for UKOOA Trenching Guidelines JIP, July 1996.

Trevor Jee Associates, 1998. *Guidelines for Trenching*. Report number SH01R17A., for UKOOA Trenching Guidelines JIP, September 1998.

US Environmental Protection Agency (EPA), 1989. *Interim procedures for estimating risks associated with exposures to mixtures of chlorinated dibenzo-p-dioxins and di-benzofurans (PCDDs and PCDFs) and 1989 update*. Prepared by risk assessment forum, EPA Washington, DC, EPA 626/3-89/016

Vellinga, T., 1989. *Land Based Disposal in the Netherlands / Case Study*. Proceedings of the International Seminar on the Environmental Aspects of Dredging Activities, Nantes, November 27 – December 1, 1989.

Wilkinson, M., Fawell, J.K., Whitehouse, P. and Sutton, A., 1997. *Proposed Environmental Quality Standards for Nonylphenol in water*. R&D Technical Report P42. Foundation For Water Research.

APPENDIX A CONSOLIDATION TESTING OF DRILL CUTTINGS FROM BERYL AND EKOFISK SITES

NOTE: This Appendix presents the report prepared by P.R.I.S., Reading University on consolidation tests undertaken during the late stages of Phase 2 of the JIP. This work was not undertaken by the Task 5B contractors and it is repoduced here for information at the request of UKOOA/DNV. The report is presented 'as received' without modification except for some necessary formatting adjustments and the addition of page numbers.

CONSOLIDATION TESTING OF DRILL CUTTINGS FROM BERYL AND EKOFISK SITES

D.M.Thornley (September 2001)

P.R.I.S
Reading University
Whiteknights
Reading RG6 6AB

Contents

Text

Tables

Figures

Appendices

1. Introduction

Two samples of drill cuttings from the Beryl and Ekofisk sites were received at PRIS (Reading University) on the 9th July 2001 for consolidation testing. Approximately 7 litres of material from each site was supplied in sealed plastic containers. These were immediately stored in a refrigerator at 4^0C.

A specification was supplied for sample preparation, consolidation testing and fluid sampling. This is included in Appendix 1. It was originally intended that all the fluid samples from various stages of the consolidation test along with the original sediment samples were to be sent away for chemical analyses.

However, as many samples were collected and PRIS has the facilities to carry out some of the analyses, it was considered prudent to do some initial testing at Reading University for total, inorganic and organic carbon in the fluids expelled. Total organic carbon concentrations in the drill cutting sediments were also determined.

The Ekofisk sediment was considerably finer-grained than the Beryl sediment. The quantity of material above and below 425 μm was determined by wet sieving and a full particle size analysis was carried out on the <425 μm size fractions using a Coulter LS230 laser granulometer. The results were combined to give a size analysis of the whole sediments.

On unloading the consolidation cell, shear-strength measurements were made at Reading University with a shear vane. Sediment samples were also taken in 38 mm diameter tubes for shear strength testing by Soil Mechanics Ltd, Doncaster. The fluid samples were sent to Severn Trent laboratories for chemical analyses.

2. Description of the modification to the Rowe Cell

In the standard Rowe Cell consolidation test, the fluid expelled passes through a porous plate and then to a single drainage tube in the centre of the cell top plate. For the tests described in this report a modified base plate is used that has four drainage ports, each with an on/off valve.

One port is positioned centrally and the other three ports are positioned equidistant, 7 cm outwards from the centre of the plate. A web-like series of shallow grooves links these drainage ports to allow the fluid expelled to reach the sample collection ports more quickly.

When the sample has been consolidated at the initial "bedding in" pressure, the cell is turned over and the modified Rowe Cell base-plate then becomes the top plate through which the fluid is expelled.

3. Initial tests prior to sample preparation

The specification for this work required that the drill-cutting sediments be made up to a moisture content equivalent to 1.5 times the liquid limit (LL) with simulated seawater, prior to loading into the Rowe Cell. The simulated seawater composition was nominally 35,000 ppm total salts in de-ionised water, as shown in Table 1.

Salt	g/l
NaCl	27.47
$CaCl_2$	1.136
KCl	0.777
$MgSO_4.7H_2O$	7.472
MgCl. $6H_2O$	5.218

Table 1. Composition of simulated seawater

The cone penetrometer method was used for the liquid limit tests. However, this method specifies that the material used should be <425 μm. Material >425 μm would inhibit the fall of the cone into the sediment and give an incorrect LL value. The <425 μm fraction has the major influence on the LL properties of the sediment.

The samples for LL determination were prepared by wet sieving through a <425μm sieve using de-ionised water. These were then reduced to a slurry by evaporating most of the liquid in a fan-assisted oven at 60^0C.

4. Results of the initial tests

Wet Sieving

The Beryl sample contained 51.4% of material <425 μm and the Ekofisk sample 91.3% of material <425 μm.

Moisture content

The moisture content was determined on the "as received" samples according to British standard BS 1377 Part 2 (Beryl 29.14 % and Ekofisk 52.34%).

Liquid limit

A cone penetrometer liquid limit test was carried out on the <425 μm fraction according to British standard BS 1377 Part 2. The liquid limit of the Beryl and Ekofisk <425 μm size fractions are 29.8% and 48.1% respectively.

5. Sample preparation

Beryl

It was requested that the Beryl sample be tested first. Approximately 4 litres of the "as received" Beryl sample (moisture content 29.14%) was prepared. As Beryl contains a large amount of coarse material (48.6% of the size fraction >425μm), it was considered that adding sea water to take the sediment up to 1.5 times the liquid limit would make it too sloppy.

Therefore the sediment was mixed and homogenised with seawater and brought to a consistency that was considered to be suitable. This was judged from previous experience with sediments of this type. The moisture content of the prepared material was 37.19%, this being 1.25 times the LL value obtained on the <425 μm sample.

Ekofisk

Approximately 4 litres of the "as received" Ekofisk sample was prepared by adding simulated sea water to bring up to 1.5 times the LL obtained on the <425 μm sample. As this sample contained only 8.9% of material >425 μm it was felt that an adjustment of the quantity of sea water to be added was not necessary. The sample was thoroughly homogenised.

6. Sample Loading and Rowe Cell Assembly

Great care was taken to ensure that no air was trapped during all stages of sample loading and cell assembly. Full details of the procedure are given in Appendix 1.

7. Initial "bedding in" of the sample

An initial consolidation pressure of 10.3kPa was set and the sample was allowed to consolidate for 3 days at this "seating pressure". All valves were then shut. The cell was then turned over so that the drainage path would now be through the modified Rowe Cell base that has 4 drainage outlets. This now becomes the top Rowe Cell plate.

8. Consolidation Tests

The consolidation pressure was increased to 18.6 kPa. Vials were positioned at the four drainage tubes to collect the expelled fluid.

The dial gauge reading was noted and the first consolidation stage was started by applying the consolidation pressure to the sediment and opening the four drainage valves. Dial-gauge readings in the initial stages were taken at intervals of 0.5, 1, 4, 8,16, 32 minutes. As the test continued, readings were taken at less ordered time intervals.

When the secondary-compression stage was judged to be almost complete, all valves were shut and the consolidation pressure was increased to 39 kPa. The consolidation pressure valve and four sample cell drainage valves were opened and consolidation dial gauge readings and fluid samples were taken once again.

Two further consolidation stages were carried out at pressures of 78 kPa and 160 kPa. Between eight to ten fluid samples were taken at each consolidation stage. These were immediately stored at 4^0C for subsequent chemical tests. A list of the fluid samples collected is given in Tables 2 & 3.

1st Pressure increment
Previous pressure: 9.3 kPa
Test Pressure 19.6 kPa

Liquid sample ident	Vol liquid collected cc	Cum volume collected cc	Collection started	Collection finished	Time to collect min
1	14	14	0	2.5	2.5
2	7	21	2.5	4	1.5
3	12.5	33.5	4	16	12
4	4.5	38	16	32	16
5	2	40	32	48	16
6	1	41	48	64	16
7	0.3	41.3	64	81	17
8	0.4	41.7	81	150	69

3rd Pressure increment
Previous pressure: 39 kPa
Test Pressure 78 kPa

Liquid sample ident	Vol liquid collected cc	Cum volume collected cc	Collection started	Collection finished	Time to collect min
18	23	23	0	1.5	1.5
19	14	37	1.5	3.58	2.08
20	5	42	3.58	6	2.42
21	2.5	44.5	6	10	4
22	5	49.5	10	16	6
23	5	54.5	16	36	20
24	2	56.5	36	60	24
26	12.5	69	60	5760	5700

2nd Pressure increment
Previous pressure: 19.6 kPa
Test Pressure 39 kPa

Liquid sample ident	Vol liquid collected cc	Cum volume collected cc	Collection started	Collection finished	Time to collect min
9	32.5	32.5	0	1.57	1.57
10	28	60.5	1.57	4	2.43
11	10	70.5	4	6	2
12	12	82.5	6	12	6
13	4	86.5	12	16	4
14	7	93.5	16	36	20
15	5.5	99	36	69	33
17	0.7	99.7	69	4362	4293

4th Pressure increment
Previous pressure: 78 kPa
Test Pressure 160 kPa

Liquid sample ident	Vol liquid collected cc	Cum volume collected cc	Collection started	Collection finished	Time to collect min
27	14	14	0	0.433	0.433
28&29	18	32	0.433	0.92	0.487
30	7.5	39.5	0.92	2.31	1.39
31	5	44.5	2.31	5	2.69
32	4	48.5	5	7.51	2.51
33	7	55.5	7.51	18.49	10.98
34	4	59.5	18.49	37	18.51
36	17	76.5	37	202	165
37	(25cc collected at increased pressure of 162kPa)				

Table 2. List of fluid samples collected from Beryl consolidation experiments

Ist Pressure increment
Previous pressure: 9.3 kPa
Test Pressure: 19.6 kPa

Vial identification	Vol liquid collected	Cum volume collected	Collection started	Collection finished	Time to collect
	cc	cc	min	min	min
E1	19	19	0	8	8
E2	19	38	8	30	22
E3	16	54	30	60	30
E4	12	66	60	90	30
E5	20	86	90	157	67
E6	15	101	157	225	68
E7	12	113	225	300	75
E8	8	121	300	360	60
E9	13	134	360	500	140
E10	21	155	500	1368	868
E11	4	159	1368	2798	1430

3rd Pressure increment
Previous pressure: 39 kPa
Test Pressure: 78 kPa

Vial identification	Vol liquid collected	Cum volume collected	Collection started	Collection finished	Time to collect
	cc	cc	min	min	min
E40	18	19	0	8	8
E41	18	37	8	30	22
E42	18	55	30	64	34
E43	18	73	64	120	56
E44	19	92	120	210	90
E45	15	107	210	345	135
E46	24	131	345	1425	1080
E47	4	135	1425	2875	1450

2nd Pressure increment
Previous pressure: 19.6 kPa
Test Pressure: 39 kPa

Vial identification	Vol liquid collected	Cum volume collected	Collection started	Collection finished	Time to collect
	cc	cc	min	min	min
E20	18	18	0	4	4
E21	18	36	4	16	12
E22	13	49	16	30	14
E23	21	70	30	60	30
E24	16	86	60	90	30
E25	13	99	90	120	30
E26	20	119	120	180	60
E27	16	135	180	240	60
E28	12	147	240	300	60
E29	17	164	300	420	120
E30	30	194	420	1659	1239
E31	3	197	1659	4299	2640

4th Pressure increment
Previous pressure: 78 kPa
Test Pressure: 160 kPa

Vial identification	Vol liquid collected	Cum volume collected	Collection started	Collection finished	Time to collect
	cc	cc	min	min	min
E60	19	19	0	8	8
E61	18	37	8	30	22
E62	17	54	30	64	34
E63	18	72	64	120	56
E64	12	84	120	183	63
E65	13	97	183	290	107
E66	10	107	290	421	131
E67	17	124	421	1426	1005
E68	3	127	1426	2851	1425

Table 3. List of fluid samples collected from Ekofisk consolidation experiments

9. Unloading the Rowe Cell

The consolidation pressure was reduced to zero and all valves were then closed. The cell was turned over and the top plate with bellows was removed. The thickness of the consolidated material was determined with callipers and sediment samples were taken for moisture content determination.

Samples were also collected for triaxial testing to measure the drained shear strength parameters. These were collected in thin-walled 38 mm diameter tubes, sampling the material several times so as to build up a continuous sample. Each tube contained sediment approximately 120 to 160 mm in length and three tubes were collected for each sample. Microcrystalline wax was poured into the tube ends to preserve the samples in the saturated state and end caps were put in place. These samples were then sent to Soil Mechanics Ltd. for testing.

A field shear vane was used to measure the undrained shear strength on the undisturbed sample left in the Rowe cell. Sediment was also saved for subsequent chemical analysis. All the drill cutting material collected was immediately stored at 4^0C.

10. Results of the consolidation experiments and associated tests

From the consolidation tests

Consolidation data for the four loading increments are shown in Tables 4 & 5. The square- root time curve fitting method devised by Taylor was used to analyse the data. The t_{90} value from the laboratory curve corresponding to 90% of primary consolidation was used to calculate the coefficient of consolidation (c_v). The methods of calculation are given in BS 1377 Part 5.

The voids ratio was calculated for the material at the end of the test. It was then possible to calculate the void ratio (e) at the start of each load increment. An assumed grain density of 2.65 Mg/m^3 was used. The coefficient of volume compressibility (m_v) and the coefficient of permeability (k) were calculated.

Plots of c_v, $m_{,v}$, k and e versus consolidation pressure are also given.

				Consolidation test calculations for square root time method							
				Voids ratio change factor		0.02462					
				Height of solids (mm)		40.616					
				Initial voids ratio (at10.3kPa)		0.7481					
			Tv ~ Theoretical time factor for T90			0.848					
Increment number	Pressure	Settlement	Cumulative Settlement	Consolidated height	Change in voids ratio	Voids ratio	Coefficient volume compressibility	Time for 90% of primary consolidation	Average sample thickness	Coefficient of consolidation	Coefficient of permeability
	p		dH	H1	de	e	mv	t90	H	Cv	k
	kPa		mm	mm			m2/MN	min	mm	m2/year	m/s
0	10.3		0	71.000	0.0000	0.7481					
1	19.6	0.71	0.71	70.290	0.0175	0.7306	1.07527	19.2	70.645	115.854	3.87E-08
2	39	1.57	2.28	68.720	0.0561	0.6920	1.15134	4.41	69.505	488.253	1.75E-07
3	78	0.79	3.07	67.930	0.0756	0.6725	0.29477	6.5	68.325	320.108	2.93E-08
4	160	0.91	3.98	67.020	0.0980	0.6501	0.16337	4.75	67.475	427.212	2.17E-08

Table 4. Geotechnical parameters calculated from the Beryl consolidation experiments

				Consolidation test calculations for square root time method							
				Voids ratio change factor		0.04771					
				Height of solids (mm)		20.960					
				Initial voids ratio (at10.3kPa)		1.4199					
			Tv ~ Theoretical time factor for T90			0.848					
Increment number	Pressure	Settlement	Cumulative Settlement	Consolidated height	Change in voids ratio	Voids ratio	Coefficient volume compressibility	Time for 90% of primary consolidation	Average sample thickness	Coefficient of consolidation	Coefficient of permeability
	p		dH	H1	de	e	mv	t90	H	Cv	k
	kPa		mm	mm			m2/MN	min	mm	m2/year	m/s
0	10.3		0	50.720	0.0000	1.4199					
1	18.6	3.12	3.12	47.600	0.1489	1.2710	7.41135	324	49.160	3.325	7.66E-09
2	39	3.83	6.95	43.770	0.3316	1.0883	3.94422	286	45.685	3.253	3.99E-09
3	78	2.59	9.54	41.180	0.4552	0.9647	1.51726	166	42.475	4.844	2.29E-09
4	160	2.67	12.21	38.510	0.5826	0.8373	0.79070	94	39.845	7.528	1.85E-09

Table 5. Geotechnical parameters calculated from the Ekofisk consolidation experiments

Undrained shear strength

On unloading the Rowe Cell an ELE field shear-vane was used to determine the undrained shear strength. The Beryl sample gave a value of 10.8 kN/m^2 and the Ekofisk 26.7 kN/m^2.

Moisture Content

The moisture content of the sediments in the "as received" state, on loading and unloading from the Rowe cell are given below. The moisture content at 1.5 times the liquid limit is also given for reference.

	Beryl	Ekofist
As received	29.14	52.34
At 1.5x the liquid limit (Calculated)	44.7*	72.15*
Seawater added	37.19%	73.44%
After consolidation at 160 kPa	24.53%	31.60%

* Calculated on the liquid limit value of the<425µm size fraction.

Table 6. Moisture content of the sediments at various stages.

11. Particle size analyses

Wet sieving showed that the Beryl sample contained 51.4% of material <425 µm and the Ekofisk sample 91.3% of material <425 µm.

Particle size analysis of the <425 µm size fractions of both sediments was carried out using a Coulter LS 230 laser granulometer. Data from the two techniques are combined to give a particle-size analysis of the sediments. The results are given in Table 7.

Class range (µm)	Beryl	Ekofisk
	%	%
<2 (clay size fraction)	7.0	17.3
2 – 63 (silt size fraction)	22.6	60.0
63–425 (v.fine, fine, medium sands)	21.8	14.0
>425	48.6	8.7

Table 7. Particle size analysis results

The Ekofisk sample contains considerably more clay and silt size fractions than the Beryl sample.

12. Total carbon, inorganic and organic carbon analysis of the expelled fluid

A Shimadzu TOC 5000 instrument was used for the analyses of total and inorganic carbon in a fluid sample taken half way through the consolidation stage at each pressure increment for both the Beryl and Ekofist samples. The results are shown in Table 8. The instrument measures total and inorganic carbon content, and the organic carbon content is determined by difference. The inorganic carbon would be mainly present in the carbonate minerals.

Fluid sample	Pressure Increment	Test pressure (kPa)	Total carbon (ppm)	Inorganic carbon (ppm)	Organic carbon (ppm)
Beryl 3	1st	19.6	89	24	65
12	2nd	39	90	26	64
22	3rd	78	90	23	67
31	4th	160	85	21	64
Ekofisk E4	1st	19.6	48	11	37
E23	2nd	39	54	11	40
E43	3rd	78	51	10	41
E63	4th	160	50	10	40

Table 8. Total carbon, inorganic and organic carbon analysis of the expelled fluids.

13. Total organic carbon content of the Beryl and Ekofisk sediments

The Walkley-Black method modified by Jackson was used to determine the total organic content of the drill cutting sediments. The results are shown in Table 9. The method utilises exothermic heating and oxidation of the sediment with potassium dichromate and concentrated sulphuric acid. The solution is then back titrated with ferrous ammonium sulphate solution.

Sediment from both Beryl and Ekofisk were analysed in the three states. These were the "as received" state, the "prior to consolidation" state (seawater added) and the "after consolidation" state.

It was thought that the high chloride concentrations could perhaps interfere with the TOC results. Therefore a second set of samples in which most of the salt had been removed were analysed for TOC. They were prepared by shaking a portion of the sample in de-ionised water for one hour followed by centrifuging. The clear fluid was decanted off and the process was repeated.

	TOC % by weight (unwashed)	TOC % by weight (washed)
Beryl "as received"	0.69	0.64
prior to consolidation	0.68	0.58
after consolidation	0.63	0.73
Ekofisk as received	1.8	1.77
prior to consolidation	2.58	2.46
"after consolidation"	2.55	2.43

Table 9. Total organic carbon content of the drill cutting sediments

The TOC content of the Ekofisk "as received" sample is approximately 30% lower than that obtained on the sample prepared and homogenised for the consolidation test. The "as received" sample was not taken at the same time. Four litres of material were used in the consolidation test, and the value for TOC of 2.5 % would therefore be a more reliable assessment.

14. Conclusions

Both sediments were extremely heterogeneous in the "as received" state. The moisture content determinations on the "as received" samples show that they were both very near the liquid limit.

The two samples were very different, however. Particle-size analysis shows the Beryl sediment to be a silty sand with some fine-to-medium gravel. A small amount of clay-size fraction is present.

The Ekofisk sample is much finer, being predominately silt with an appreciable amount of clay size fraction. There is some sand present, with a small amount of fine gravel.

All the consolidation tests that were performed went well .The tests showed that the Beryl sample is much stiffer and more permeable than that for Ekofisk. Over the stress range of the tests (10 to 160 KPa) the Beryl compressibility (mv) decreases from 1.07 to 0.16 m2/MN and the coefficient of consolidation (Cv) increases from 116 to 427 m2/yr.

The Ekofisk sample showed similar trends with mv decreasing from 7.4 to 0.79 m2/MN and Cv increasing from 3.3 to 7.5 m2/yr. The derived coefficients of permeability of the Beryl sample are some 6 to 15 times greater than those for Ekofisk, reflecting the lower clay content of the Beryl sample.

Undrained shear strength measurements were determined at Reading on sediment at the end of the consolidation experiment. The undrained shear strength of the Ekofisk sediment was 2.5 times greater than the Beryl. Material in the consolidated state was also collected in tubes for drained shear-strength measurements. These were sent to Soil Mechanics, Ltd for analysis and the results will be reported by them directly to DNV.

Preliminary analyses were also performed at Reading on some of the expelled fluid samples and the sediments. The total hydrocarbon concentration in the expelled pore fluids does not change significantly throughout the four pressure increments.

The concentration of total organic carbon in the Beryl drill cuttings was 0.66% and the Ekofisk, 2.52%. However it is interesting to note that the although the TOC concentration in the Beryl drill cuttings was lower than that in the Ekofisk, the concentration of organic carbon contained in the fluids expelled was higher in the Beryl (65 ppm) when compared to the Ekofisk (40ppm).

This could be explained by examining the particle-size distribution of the two sediment samples. The Ekofisk sediment has a much higher silt and clay content and it is suggested that this finer-grained material retains the oil more strongly.

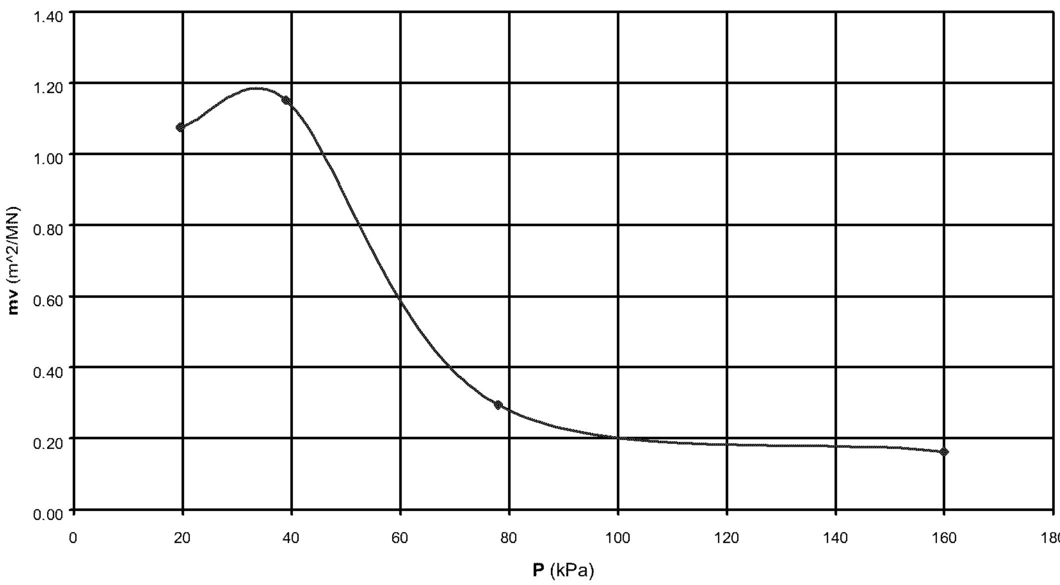


Figure 1. Beryl Coefficient of Volume Compressibility vs Consolidation Pressure

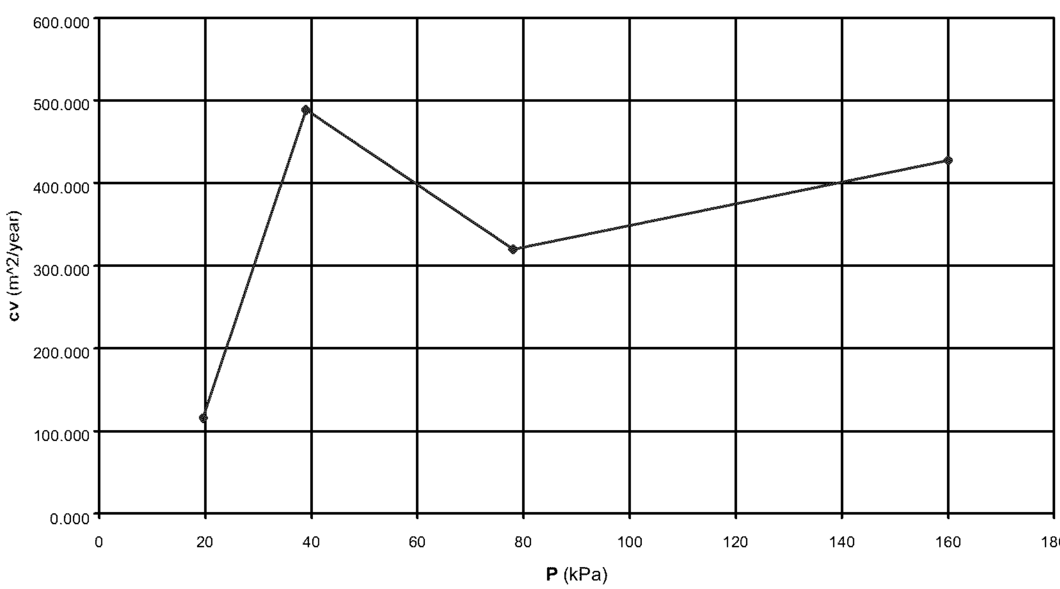


Figure 2. Beryl Coefficient of Consolidation vs Consolidation Pressure

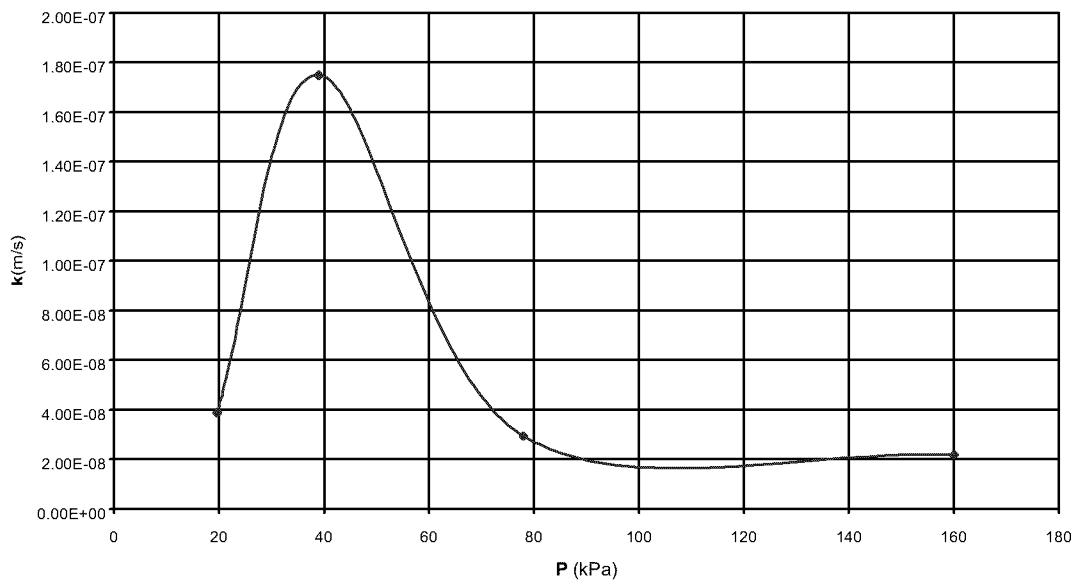


Figure 3. Beryl Coefficient of Permeability vs Consolidation Pressure

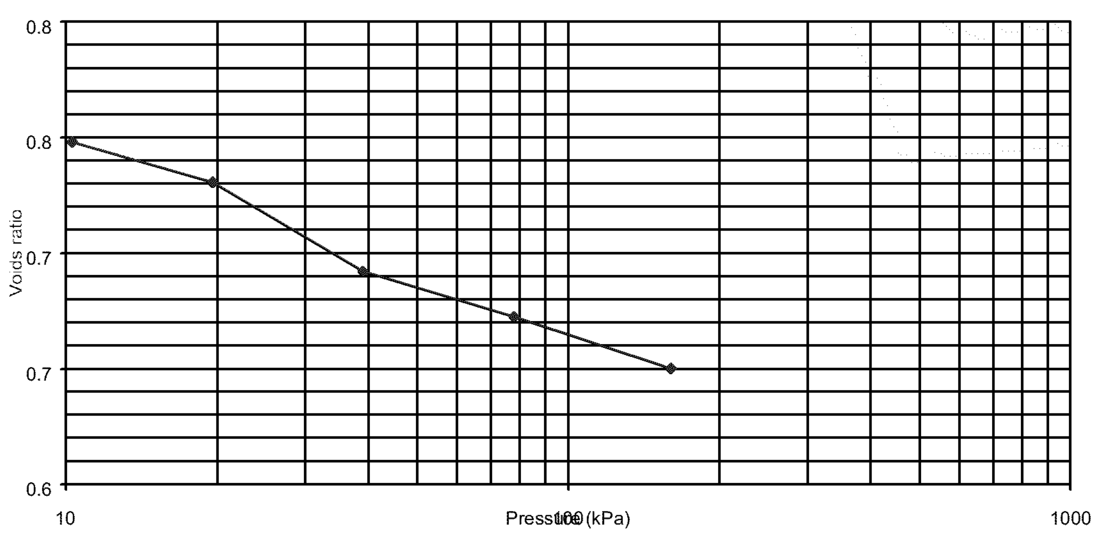


Figure 4. Beryl Voids Ratio vs Consolidation Pressure

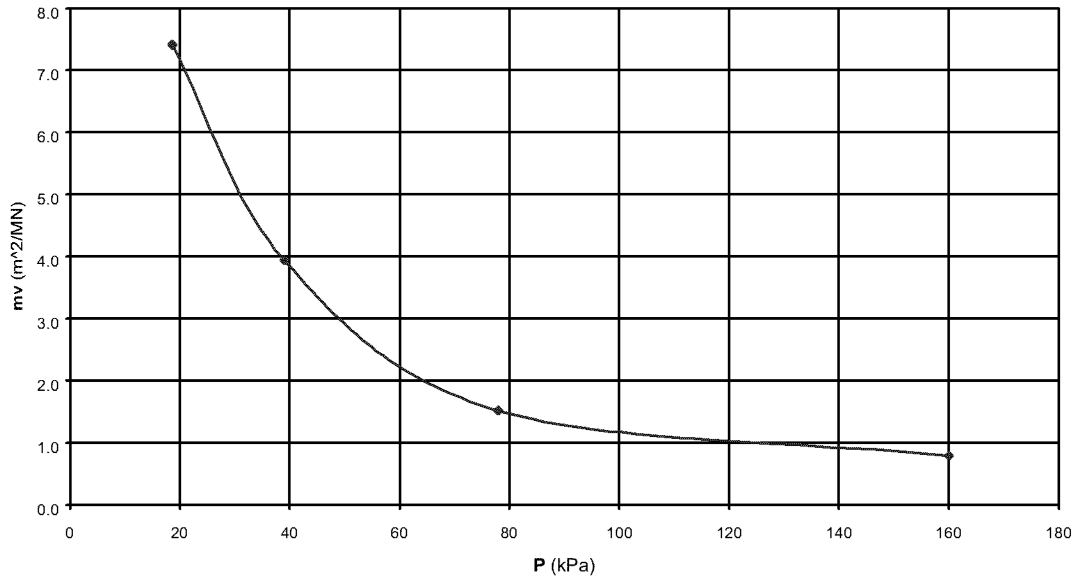


Figure 5. Ekofisk Coefficient of Volume Compressibility vs Consolidation Pressure

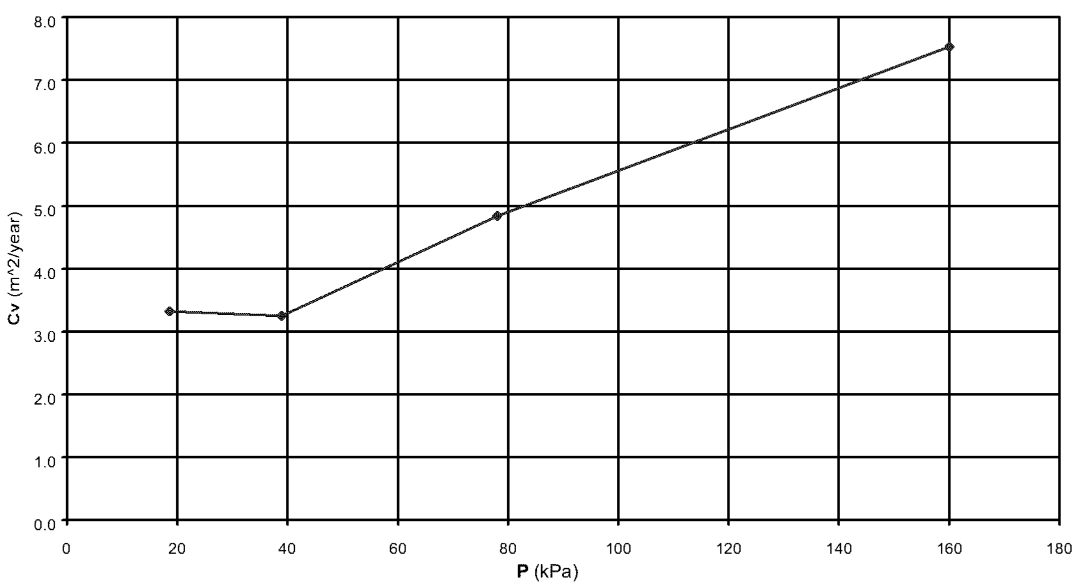


Figure 6. Ekofisk Coefficient of Consolidation vs Consolidation Pressure

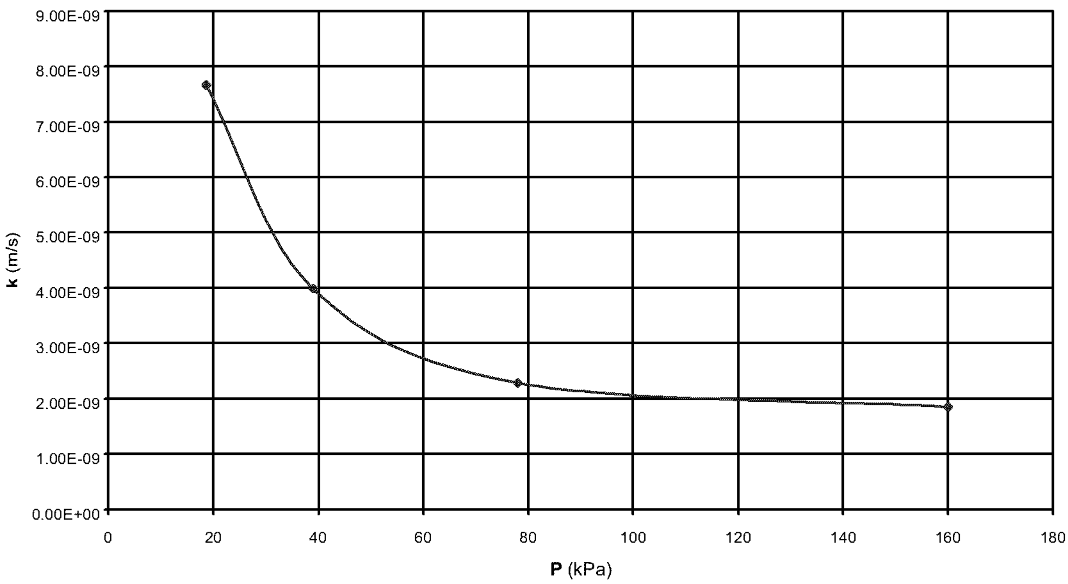


Figure 7. Ekofisk Coefficient of Permeability vs Consolidation Pressure

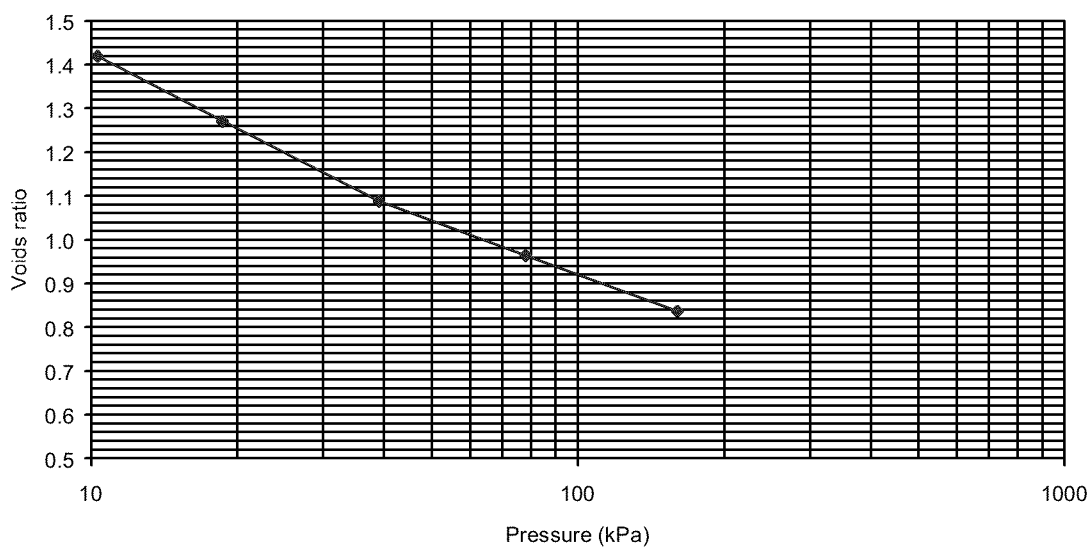


Figure 8. Ekofisk Voids Ratio vs Consolidation Pressure

Appendix 1

Specification for Consolidation Testing

For the UKOOA Drill Cuttings Initiative

Postgraduate Research Institute In Sedimentology (PRIS) University of Reading

<u>Material</u> – Two samples of cuttings (approximately 7 litres of each) in sealed containers have been provided to PRIS. These are from the Beryl and Ekofisk sites. The samples are clearly labelled.

<u>Sample preparation</u>

Determine the liquid limit (LL) for both samples.

Homogenise approx 5000 ml of sample with sufficient quantity of sea water (or "made up" brine 30,000 ppm) to approx 1.5x LL as necessary.

Preserve a quantity of homogenised material under refrigeration for future chemical analysis.

Determine the moisture content of the original and homogenised sample.

<u>Consolidation Testing</u> – Consolidate the homogenised sample in a hydraulic consolidation cell, modified to give representative samples of pore fluid expelled during the consolidation process. Such pore fluid should be stored under refrigeration to prevent degredation.
Then proceed as follows:

"Bed in" the sample under a low consolidation pressure (approx 10 kN/m^2)

Apply consolidation pressure in four increments - approx 10-20, 20-40, 40-80, 80-160 kN/m^2

For each increment use the volume change and its rate to calculate the coefficient of volume compressibility and the coefficient of consolidation.

During each consolidation stage collect samples of pore fluid; a minimum of 10 samples should be collected. The time at which the sample collection is be made should be recorded on the sample container, together with other data defining the sample.

<u>Samples for Shear Strength Measurement</u> – on completion of the final load increment, dismantle the apparatus, taking care not to wet the consolidated sample. Using a thin walled cutting tube (38mm dia) sub-sample the cuttings material several times so as to build up a continuous sample with a thickness greater than 35mm.

Three such samples should be prepared. The ends of the samples should be sealed to prevent water loss, and the samples should be stored in cool conditions. A final moisture content for the sample should be measured and a sample of the consolidated material refrigerated for future chemical analysis.

<u>Testing Pore Fluid and Cored Samples</u> – PRIS will not be required to carry out chemical analysis of the pore fluids or consolidated material or to undertake shear strength testing, but should preserve the samples for analysis by commercial laboratories.

Reporting – PRIS will prepare a brief report describing the procedures used in sampling and testing, list the samples collected and present a table of the consolidation test results.

Programme – The Beryl sample should be tested first and it is hoped that the results should be available by the end of September 2001. It is however recognised that at this stage the duration of each consolidation stage will depend on the properties of the samples.

Appendix 2

Sample Loading and Rowe Cell Assembly

To avoid trapping any air in the Rowe Cell, all stages of sample loading and cell assembly were carried out according to the following procedure. The base of the cell was covered with a thin film of de-aired sea water and a porous plate saturated with de-aired sea water was lowered to the cell base. The Beryl sample was loaded but not de-aired. The surface was levelled and the depth of the sample was measured with callipers.

The Ekofist sediment was de-aired in five layers. Approximately 1/5 of the sample was transferred into the cell and a domed desiccator lid with an "O"ring seal placed on top. The sample was de-aired for 5 minutes using a rotary pump. Air was then admitted and the next portion of the sample was then added to the cell and de-aired. The remaining three portions were de-aired in this manner.

The sediment was covered with a thin film of de-aired sea water and a porous plate saturated with de-aired de-ionised water was placed on top. De-ionised water was added to cover the porous plate and the rigid aluminium loading plate was placed on top. The top of the Rowe cell with the diaphragm and central hollow rod drainage system section and central hollow rod drainage system was then put in place. The inside of the Rowe cell was then almost filled with de-ionised water

Using two-inch thick spacer blocks, the top metal plate of the Rowe Cell was raised and supported above the cell body. The inside of the diaphragm was then almost filled with de-aired de-ionised water. The blocks were then removed and the Rowe Cell was assembled. The inside of the diaphragm was filled with water through the water pressure valve, using the bleed valve to release any air.

A small seating pressure of 5 kPa was then applied to the diaphragm. The rim drainage valve was opened to remove excess water above the sample and release air that may be trapped around the outside of the diaphragm.

All valves were then shut and the compression dial gauge was fitted. An initial consolidation pressure of 10.3 kPa was set and the diaphragm pressure valve and sample cell drainage valves were opened. The sample was allowed to consolidate for 3 days at this "seating pressure". All valves were then shut prior to turning the cell over for the start of the consolidation tests.

APPENDIX B SAMPLE ANALYSIS METHODS

B1 TOXICITY TEST METHODS

Samples of cuttings water were removed from the sample container in a solvent-cleaned glass beaker so as to prevent contamination of the cuttings water with plasticisers (such as phthalate esters) which may leach from plastic containers. The settled cuttings water was diluted using clean filtered seawater to produce a dilution series for toxicity testing. In this way the amount of dilution required to remove the toxic effects of the cuttings water could be determined. The dilution series included 100% clean seawater and 100% cuttings water and five dilutions containing 3,6,10,33 and 56% cuttings water mixed with clean seawater. In order to confirm that the test organism (the crustacean Tisbe battaglai) from the laboratory culture were in good condition, a positive control toxicity test series was also conducted using zinc sulphate, a toxicant with known and predictable effects. The response of the test organism in this test can therefore be compared with a standard value established from a wide range of studies and laboratories, thereby validating the test if the value falls within the expected range. Within this study the control response met the acceptability criteria for the test.

B2 HYDROCARBONS ANALYSIS

Samples of leachates from the cuttings samples were removed from the sample container in a solvent-cleaned glass beaker so as to prevent contamination from plasticisers, such as phthalates, present in all plastics. The leachates were filtered through 0.45µm regenerated cellulose filters, and extracted into n-pentane prior to analysis using coupled gas chromatography/mass spectrometry. This is a powerful analytical technique that allows the identification and quantification of a wide range of constituent compounds including both aliphatic and aromatic hydrocarbons. Individual PAH components were quantified using a well-tested and validated procedure (Kelly et al., 2000), which involves the use of deuterated surrogate standards added at known concentrations prior to extraction. Total hydrocarbon concentrations were determined by means of fluorescence spectrometry using Forties crude oil as a calibrant, and the concentrations of total n-alkanes were estimated using the sum of the mass signals observed at 57 Daltons (an ion common to all alkane compounds. In oils generally n-alkanes occur as an homologous series of prominent peaks in the chromatograms (Appendix II). Sulphur (S8) concentrations were estimated in a similar manner, using the two major diagnostic ions at 64 and 192 Daltons. In both cases the signals were related to that obtained for squalane, which was added as an internal standard so that the concentrations of n-alkanes and sulphur could be estimated.

B3 EXTRACTION AND ANALYSIS OF TRACE METALS

Seawater samples were acidified on collection and filtered through a 0.4um Nuclepore polycarbonate membrane. The method uses a complexation technique with liquid/liquid extraction both to remove the trace metals from the interfering effects of the major 'salt' elements, and to concentrate the trace elements prior to analysis (Jones and Laslett, 1994). The metals are extracted from the water into 1, 1, 1-trichlorotrifluoroethane using a mixed dithiocarbamate complexing agent. They are then back extracted into dilute nitric acid before analysis by inductively- coupled plasma mass spectrometry.

A certified reference material CASS-4, which is a coastal sea water, prepared by the National Canadian Reference Laboratory and providing certified values for each trace metal determined was analysed along side the samples to provide quality control. The determined values for all the trace metals fell within the certified range established for CASS-4, and so demonstrated the validity of the analyses.

B4 EXTRACTION OF DISSOLVED PHASE ALKYLPHENOLS

For the extraction of alkylphenol compounds [nonylphenol (NP), octylphenol (OP) nonylphenol mono-ethoxylate (NP1EO) and nonylphenol di-ethoxylate (NP2EO)], the following procedure was adopted. The aqueous sample was accurately measured and then filtered over a 0.45 μm hexane rinsed glass fibre filter. Butylphenol was added to the filtrate as a surrogate standard at a concentration of 20 ng l^{-1}. One ml of methanol was then added to pre-treat the sample and aid extraction during solid phase extraction (SPE). Following sample equilibration (30 minutes), the sample was drawn through a pre-conditioned C18 silica-bonded, solvated and pre-equilibriated SPE cartridge. After extraction, the cartridge was dried by pulling air through it, wrapped in solvent rinsed aluminium foil and stored at -20°C prior to analysis.

Cartridges were defrosted and analytes were eluted with ethyl acetate followed by dichloromethane. Eluants were combined and residual water was removed by the addition of anhydrous sodium sulphate. Sample extracts were cleaned-up using glass chromatography columns loaded with deactivated alumina topped with anhydrous sodium sulphate. Analytes were eluted with ethyl acetate, followed by dichloromethane. The combined elutriates were then reduced in volume using a gentle stream of oxygen-free-nitrogen. Extracts were stored at -20 °C prior to analysis using gas chromatography-mass spectrometry (GC-MS).

B5 ANALYSIS BY GC-MS

Sample extracts were analysed by GC coupled to an ion trap MS instrument (Finnigan MAT, California, US). The MS was operated in the positive-ion electron impact mode and in and full scan. Alkylphenols were identified according to the method described by Blackburn and Waldock (1995) by quantifying the clusters of isomer peaks as masses (expressed as Daltons)- of 107 Daltons (Da) and 135 Da NP relative to the internal (butylphenol) standard molecular ion (150 Da) or parent ion 135 Da. The ethoxylates (NP1EO and NP2EO) were quantified by integrating the cluster of isomers at 179 Da for NP1EO and 223 Da for NP2EO, respectively. Four calibration standards were analysed before the sample extracts. [

Alykylphenols were quantified relative to the internal standard after the calculation of response factors for each compound type. Concentrations of the ethoxylates were expressed as NP1EO + NP2EO compared to a working standard of Igepal CO 210. Procedural blanks consisting of a volume of tap water equivalent to that of the samples, and which had been taken through the entire analytical procedure, were included within the batch of samples. Practical limits of detection were approximately 0.1 μg (NP) l-1 and 0.6 μg (NPEO+NP2EO) l-1 in aqueous samples.